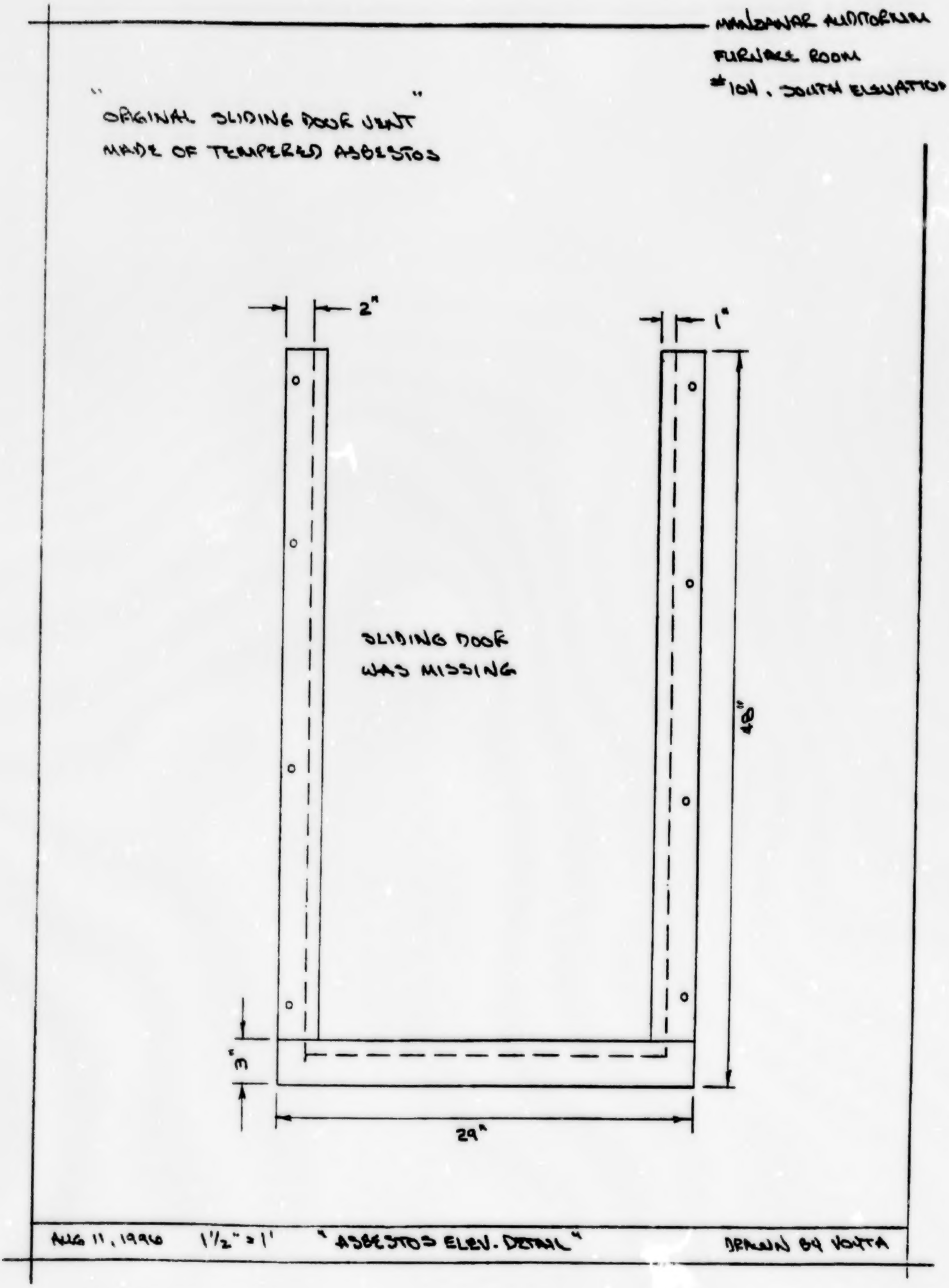

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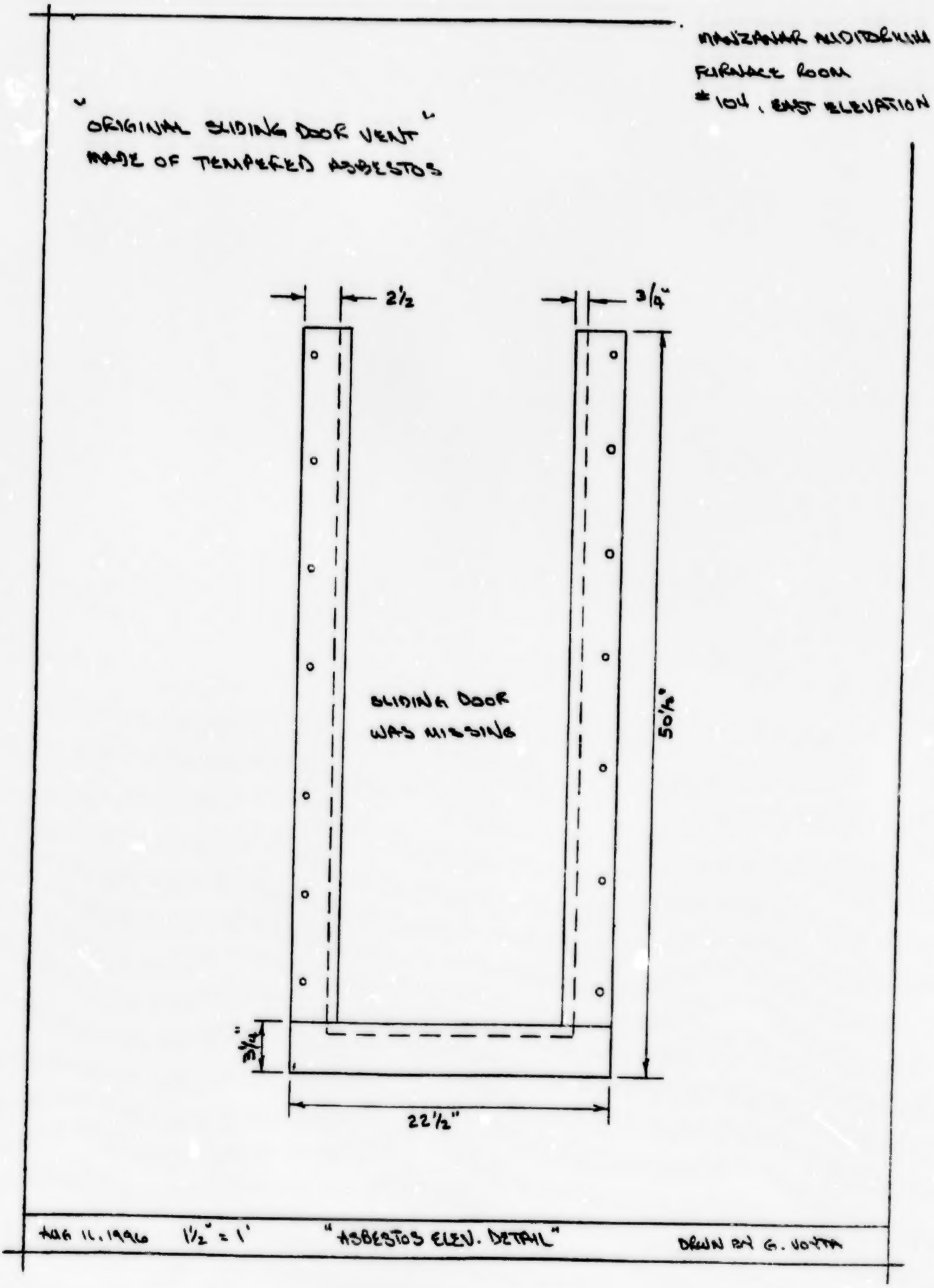

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DEVA-004-35

AUDITORIUM BUILDING

MANZANAR NATIONAL HISTORIC SITE

RM#	CEILING	N. WALL	E. Wall	S. WALL	W. WALL	FLOOR	MISC.
101	STAIR	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	OLD CONC.	CLOSET
102	GYP. BD. (PAINT)	PAINT. VERT BD.	PAINT. VERT BD.	PAINT. VERT BD.	PAINT. VERT BD.	OLD CONC.	ENTRY
103	GYP. BD.,STAIR,PAINTED. SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	OLD CONC.	TICKET OFF.
104	RNDM UNPAINTED SHIPLAP	RNDM UNPAINTED SHIPLAP	RNDM UNPAINTED SHIPLAP	RNDM UNPAINTED SHIPLAP	RNDM UNPAINTED SHIPLAP	LIN UNFIN. WOOD FLOOR	FURNACE
105	CELOTEX	PAINTED. SHIPLAP	NONE	PAINTED. SHIPLAP	PAINTED. SHIPLAP	NEW CONC.	(POOR)
106	CELOTEX (POOR)	PAINTED. SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	NEW CONC.	
107	CELOTEX (POOR)	PAINTED. SHIPLAP	CELOTEX	PAINTED. SHIPLAP	NONE	UNFIN. WOOD FLOOR	
108	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	OLD CONC., SL ON FL	JAN CL
109	PAINTED. SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	OLD CONC.	REST RM
110	PAINTED. SHIPLAP,CELOTEX,GYP. BD.,P	PAINTED. SHIPLAP,UNPAINTED SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	UNPAINTED SHIPLAP WOOD FLOOR (PAINTED)	UNFIN. WOOD FLOOR LIN	LOCKER RM
111	12x12 ACST TILE	PAINTED. SHIPLAP, CELOTEX & BATTS	PAINTED. SHIPLAP. CELOTEX & BATTS	CELOTEX, CELOTEX & BATTS	PAINTED. SHIPLAP. CELOTEX & BATTS	WOOD FLOOR, PAINTED	
112	CELOTEX P	CELOTEX	PAINTED. SHIPLAP	PAINTED. SHIPLAP	PAINTED. SHIPLAP	WOOD FLOOR, UNPAINTED	
113	CELOTEX P	PAINT. VERT BD.	PAINT. VERT BD.	PAINT. VERT BD.	PAINT. VERT BD.	WOOD FLOOR, UNPAINTED	
114	GYP. BD.	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	OLD CONC., UNPAINTED WOOD FLOOR, STAIR	STAIRWELL
115	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	OLD CONC.	BOILER RM

DEVA-004-35

AUDITORIUM BUILDING

MANZANAR NATIONAL HISTORIC SITE

RM#	CEILING	N. WALL	E. Wall	S. WALL	W. WALL	FLOOR	MISC.
116	WOOD FRAMING	PAINT. VERT BD.,GYP. BD.	WOOD FRAMING	WOOD FRAMING	VARN. WOOD FLOOR	OLD CONC.,NEW RAMP	UNDER STAGE
117	GYP. BD.	GYP. BD.	GYP. BD.	WOOD FRAMING	GYP. BD.P	OLD CONC.	
118	CELOTEX	PAINT. VERT BD.	PAINT. VERT BD.	PAINT. VERT BD.	PAINT. VERT BD.,WOOD FLOOR, PAINTED	UNPAINTED WOOD FLOOR,NEW CONC.,UNPAINTED WOOD FLOOR	STAGE
119	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING,VARN. WOOD FLOOR	OLD CONC.,NRMP	
120	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	OLD CONC.	BOILER RM
121	CELOTEX P	PAINT. VERT BD.	PAINTED. SHIPLAP	PAINT. VERT BD.	PAINT. VERT BD.	NEW CONC.	HALL
122	GYP. BD.	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	UNPAINTED WOOD FLOOR,STAIR	NEW CONC.	STAIRWELL
123	CELOTEX (POOR COND.)	PAINT. HORIZ. AND VERT BD.	PAINT. HORIZ. AND VERT BD.	PAINT. HORIZ. AND VERT BD.	PAINT. HORIZ. AND VERT BD.	NEW CONC.	AUD/GYM
201	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	OLD CONC.	PROJPROJ RM
202	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	UNPAINTED SHIPLAP	OLD CONC.	FILM REW
203	GYP. BD., PAINTED	N/A	PAINT. VERT BD.	N/A	PAINT. VERT BD.	WOOD FLOOR, (PAINTED)	STAIRWELL
204	GYP. BD., PAINTED	GYP. BD., PAINTED	GYP. BD., PAINTED	GYP. BD., PAINTED	GYP. BD., PAINTED	WOOD FLOOR, (PAINTED)	G. OFFICE
205	GYP. BD., PAINTED	GYP. BD., PAINTED	GYP. BD., PAINTED	GYP. BD., PAINTED	GYP. BD., PAINTED	WOOD FLOOR, PAINTED	G. STORAGE
206	CELOTEX	PAINT. VERT BD.	PAINT. VERT BD.	N/A	PAINT. VERT BD.	WOOD FLOOR	STAGE
207	GYP. BD., PAINTED	N/A	PAINT. VERT BD.	N/A	PAINT. VERT BD.	WOOD FLOOR, PAINTED	STAIRWELL

DEVA-004-35

AUDITORIUM BUILDING

MANZANAR NATIONAL HISTORIC SITE

RM#	CEILING	N. WALL	E. Wall	S. WALL	W. WALL	FLOOR	MISC.
208	GYP. BD., PAINTED	GYP. BD., PAINTED	GYP. BD., PAINTED	GYP. BD., PAINTED	GYP. BD., PAINTED	WOOD FLOOR, PAINTED	DRESS
209	GYP. BD.PT	PAINT. VERT BD.	GYP. BD.PT	GYP. BD.PT	GYP. BD.PT	WOOD FLOOR (PAINTED)	DR STOR
210	CELOTEX	N/A	PAINT. VERT BD.	PAINT. VERT BD.	PAINT. VERT BD.	WOOD FLOOR	STAGE
211	GYP. BD.	PAINT. VERT BD.	PAINT. VERT BD.	PAINT. VERT BD.	PAINT. VERT BD.	STAIR,WOOD FLOOR	STAIRWELL
212	GYP. BD.	PAINT. VERT BD.	PAINT. VERT BD.	GYP. BD.	GYP. BD.	STAIR	STAIRWELL
301	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FLOOR	
302	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FRAMING	WOOD FLOOR	

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APPENDIX H: MECHANICAL SYSTEMS DATA

Mechanical Systems - General

The following items represent general information and requirements pertinent to the analysis and design of mechanical systems and related building components for the Manzanar Auditorium building.

Climatic Data Climatic data used for calculating space cooling and heating loads for the Manzanar Auditorium building are as follows:

Design conditions for Independence, California (taken from ASHRAE Publication SPCDX, "Climatic Data for Region X, Arizona, California, Hawaii, Nevada") -

Latitude - 36.8°N
Elevation - 3,950'

Winter Design -

Outdoor Temperature (0.2% Design) -	19°F
Indoor Temperature (office and support spaces) -	72°F
Indoor Temperature (gym and stage) -	65°F
Heating Degree Days -	3,932

Summer Design -

Outdoor Temperature, Dry Bulb (0.5% Design) -	101°F
Mean Coincident Outdoor Wet Bulb Temperature (0.5% Design) -	60°F
Indoor Temperature (office and support spaces) -	75°F
Indoor Temperature (gym and stage) -	80°F

Domestic Water Conservation Plumbing fixtures and fittings shall be selected to comply with the following criteria (NPS Tech Bulletin 92-1 recommendations) to insure water and energy conservation:

Water Closets -	1.6 gallons per flush
Urinals -	1.0 gallons per flush
Showerheads -	2.5 gallons per minute at 80 psi
Kitchen Faucets -	2.2 gallons per minute at 60 psi
Lavatories (residential) -	2.2 gallons per minute at 60 psi

Heating and Ventilating Systems The design of the heating, ventilating, and air conditioning systems shall conform to the requirements of the 1997 Uniform Mechanical Code (UMC).

Plumbing and Domestic Water Heating Systems The design of the plumbing and domestic water heating systems shall conform to the requirements of the 1997 Uniform Plumbing Code (UPC). The building water supply shall be individually metered in accordance with NPS Staff Directive 78-10, which requires that the water service to all NPS buildings and other facilities be metered at the use point.

APPENDIX I: STRUCTURAL CALCULATIONS

Park MANZ (DEVA 004-35)	**NATIONAL PARK SERVICE** **DENVER SERVICE CENTER**		Sheet 1
Area AUDITORIUM			of
Project	By E SMITH	Checked	Pkg.
Feature	Date 9-21-98	Date	Account

LOADS (UBC 1997)

WIND - 70 mph, Exposure C

SEISMIC - ZONE 4

SNOW - 30 psf

↑ From Public Works Dept. 760-878-0209

$P_f = C_e I P_g = (0.6)(1.0)(30) = 18$ psf

ROOF LIVE LOAD - $L_r = 20$ psf

ATTIC LIVE LOAD - 10 psf

ATTIC DEAD LOAD - 10 psf (Assumed)

ROOF DEAD LOAD - 10 psf (Assumed)

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Park MANZ (DEVA 004-35)	**NATIONAL PARK SERVICE** **DENVER SERVICE CENTER**		**Sheet** 2
Area AUDITORIUM			**of**
Project	**By** E. SMITH	**Checked**	**Pkg.**
Feature	**Date** 9-22-98	**Date**	**Account**

CAPACITY OF ROOF PURLINS

ASSUME DF No. 1

$1\frac{1}{2}" \times 9"$ @ 24" o.c.

$$F_b' = F_b \cdot C_D \cdot C_M \cdot C_L \cdot C_F \cdot C_r = (1000)(1)(1)(1)(1.1)(1.15) = 1265 \text{ psi}$$

$$M = F_b' \cdot S = (1265 \text{ psi})\left(\frac{(1.5)(9)^2}{6}\right)\left(\frac{1}{12}\right) = 2135 \text{ ft}\cdot\text{lb}$$

$$M = \frac{w\ell^2}{8}$$

$$w = \frac{8M}{\ell^2} = \frac{(8)(2135 \text{ ft}\cdot\text{lb})}{(16 \text{ ft})^2} = 67 \text{ plf}$$

$$\frac{67 \text{ plf}}{2'} = \boxed{33 \text{ psf}}$$

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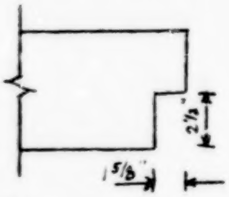
Park MANZ (DEVA 004-35)	**NATIONAL PARK SERVICE** **DENVER SERVICE CENTER**	Sheet 3
Area AUDITORIUM		of
Project	By E SMITH | Checked	Pkg.
Feature	Date 9-22-98 | Date	Account

CAPACITY OF CEILING JOIST

$1\tfrac{1}{2} \times 5\tfrac{3}{8}"$ @ 24" w/ notch



F_b :

$$R_B^2 = \frac{\ell_e d}{b^2} = \frac{[(1.63)(8')(12) + (3)(5\tfrac{3}{8})](5\tfrac{3}{8})}{(1.5)^2} = 412$$

$$F_{bE} = \frac{K_{bE}\,E'}{R_B^2} = \frac{(0.438)(1700{,}000)}{412} = 1806 \text{ psi}$$

$$F_b^* = F_b \cdot C_D \cdot C_M \cdot C_F \cdot C_r = (1000)(1)(1)(1.3)(1.15) = 1495 \text{ psi}$$

$$C_L = 0.88$$

$$F_b' = (0.88)(1495) = 1317 \text{ psi}$$

$$M = F_b' \cdot S = (1317 \text{ psi})\frac{(1.5)(5.375)^2}{6}\left(\frac{1}{12}\right) = 793 \text{ ft}\cdot\text{lb}$$

$$w = \frac{8M}{\ell^2} = \frac{(8)(793)}{16^2} = 25 \text{ plf}$$

$$\frac{25 \text{ plf}}{2'} = \boxed{12 \text{ psf}} \longleftarrow \text{CONTROLS}$$

$$F_v' = F_v \cdot C_D\, C_M = (95)(1)(1) = 95 \text{ psi}$$

$$V = \tfrac{2}{3} F_v' \cdot A = (2/3)(95 \text{ psi})(2.875)(1.5) = 273 \text{ lb}$$

$$w = \frac{2V}{\ell} = \frac{(2)(273)}{16} = 34 \text{ plf}$$

$$\frac{34 \text{ plf}}{2'} = 17 \text{ psf}$$

$$\Delta_{240} = (16)(12)/240 = 0.8$$

$$\Delta = \frac{5w\ell^4}{384EI} \rightarrow 0.8 = \frac{(5)(w)(16)^4(1728)}{(384)(1700000)\frac{(1.5)(5.375)^3}{12}} \rightarrow w = 18 \text{ psf} \ (D+L)$$

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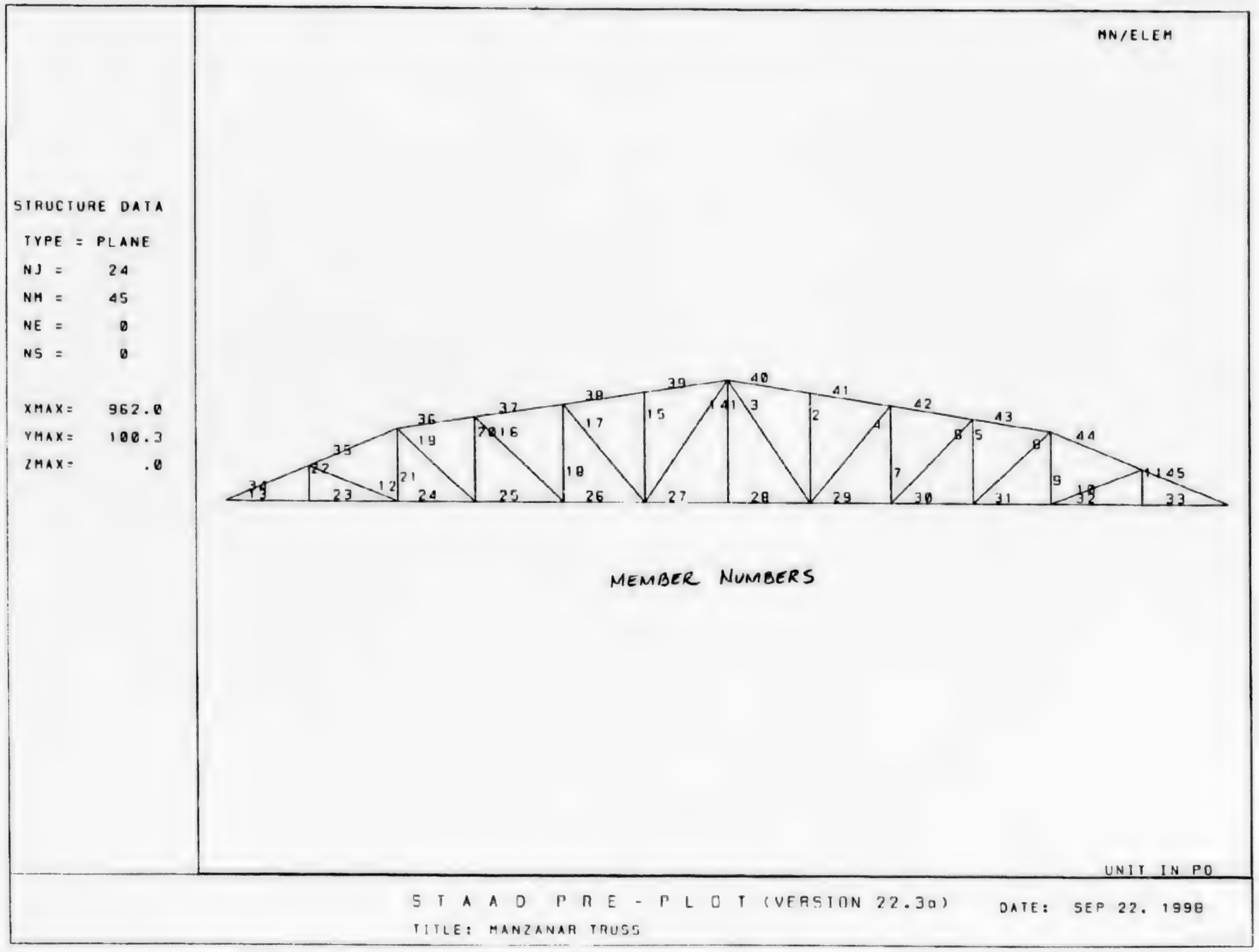


STRUCTURE DATA

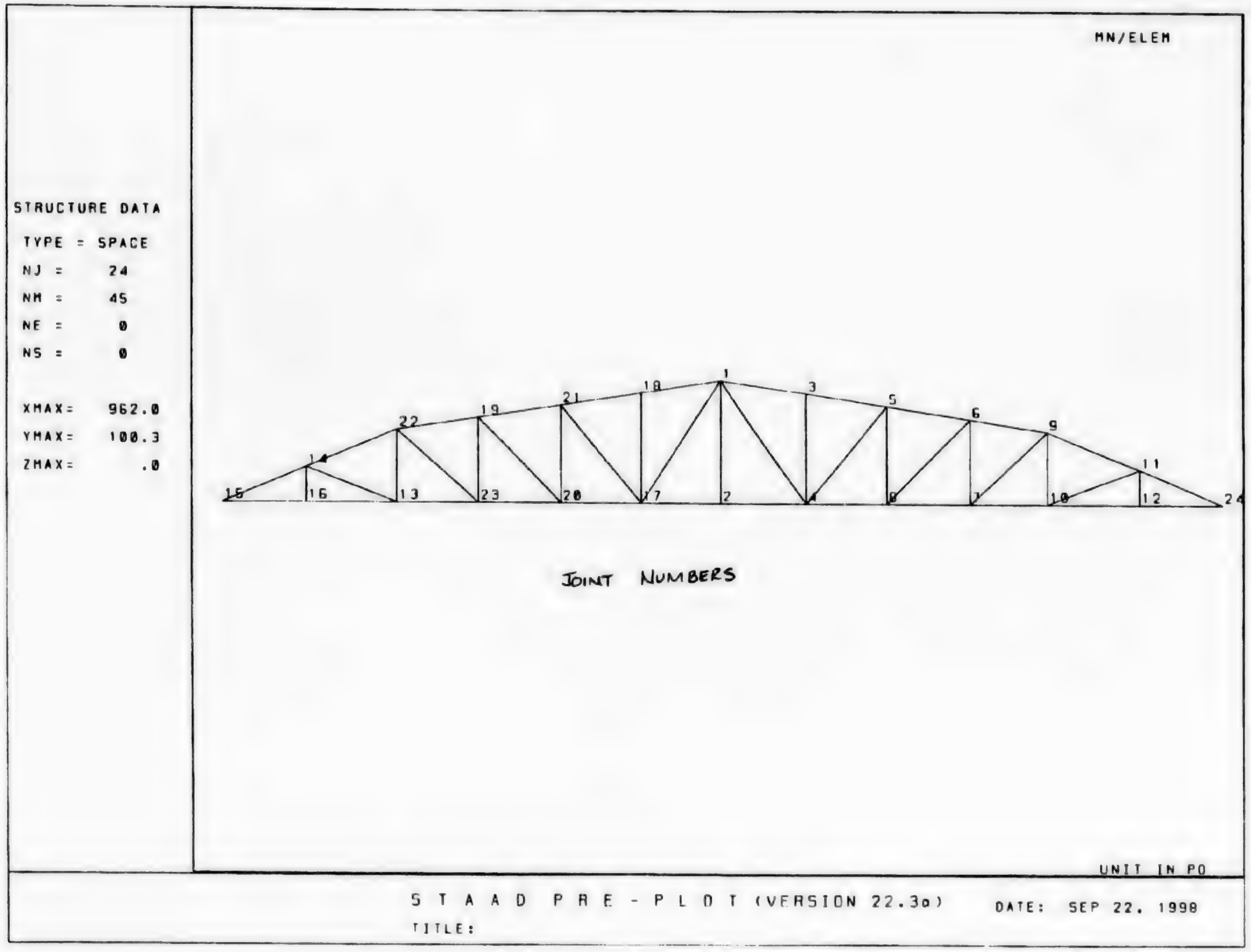
TYPE = PLANE
NJ = 24
NM = 45
NE = 0
NS = 0

XMAX= 962.0
YMAX= 100.3
ZMAX= .0

MEMBER NUMBERS

STAAD PRE-PLOT (VERSION 22.3a) DATE: SEP 22, 1998

TITLE: MANZANAR TRUSS



MN/ELEM

STRUCTURE DATA

TYPE = SPACE
NJ = 24
NM = 45
NE = 0
NS = 0

XMAX= 962.0
YMAX= 100.3
ZMAX= .0

JOINT NUMBERS

UNIT IN PO

STAAD PRE-PLOT (VERSION 22.3o) DATE: SEP 22, 1998
TITLE:

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                               truss.anl
                                              PAGE NO.    1


          ****************************************************
          *                                                  *
          *            S T A A D - III                       *
          *            Revision 22.3a                        *
          *            Proprietary Program of                *
          *            Research Engineers, Inc.              *
          *            Date=    SEP 28, 1998                 *
          *            Time=       7:46:25                   *
          *                                                  *
          *       USER ID: nps                               *
          ****************************************************


  1. STAAD PLANE MANZANAR AUDITORIUM - TRUSS
  2. *
  3. *
  4. * NOTE:  THIS ANALYSIS DOES NOT EVALUATE ALL POSSIBLE LOAD
  5. * COMBINATIONS.  IT WAS RUN AS A PRELIMINARY CHECK OF THE
  6. * STRESS IN THE TRUSS MEMBERS AND ADEQUATELY DEMONSTRATES
  7. * THAT MANY MEMBERS ARE SIGNIFICANTLY OVERSTRESSED.
  8. *
  9. *
 10. INPUT WIDTH 72
 11. UNIT FEET KIP
 12. JOINT COORDINATES
 13.       1        40.083        8.356        .000
 14.       2        40.083         .000        .000
 15.       3        46.770        7.488        .000
 16.       4        46.770         .000        .000
 17.       5        53.308        6.639        .000
 18.       6        60.037        5.765        .000
 19.       7        60.037         .000        .000
 20.       8        53.308         .000        .000
 21.       9        66.341        4.946        .000
 22.      10        66.341         .000        .000
 23.      11        73.474        2.394        .000
 24.      12        73.474         .000        .000
 25.      13        13.826         .000        .000
 26.      14         6.693        2.394        .000
 27.      15          .000         .000        .000
 28.      16         6.693         .000        .000
 29.      17        33.397         .000        .000
 30.      18        33.397        7.488        .000
 31.      19        20.130        5.765        .000
 32.      20        26.859         .000        .000
 33.      21        26.859        6.639        .000
 34.      22        13.826        4.946        .000
 35.      23        20.130         .000        .000
 36.      24        80.167         .000        .000
 37. MEMBER INCIDENCES
 38.         1         1         2
 39.         2         3         4
 40.         3         1         4
 41.         4         5         4
  MANZANAR AUDITORIUM - TRUSS                           -- PAGE NO.    2
 *
 42.         5         6         7
 43.         6         6         8
 44.         7         8         5
 45.         8         9         7
 46.         9        10         9
 47.        10        10        11
 48.        11        11        12
 49.        12        13        14
 50.        13        15        16
 51.        14         1        17
 52.        15        18        17
 53.        16        19        20
 54.        17        21        17
 55.        18        20        21
 56.        19        22        23
 57.        20        19        23
 58.        21        13        22
 59.        22        14        16
 60.        23        16        13
 61.        24        13        23
 62.        25        23        20
 63.        26        20        17
                               Page 1
```

By inspection, most top chord and bottom chord members are overstressed, and several connections are completely deficient. (See sample forces / stresses indicated.)

```
                                                truss.anl
   64.           27          17           2
   65.           28           2           4
   66.           29           4           8
   67.           30           8           7
   68.           31           7          10
   69.           32          10          12
   70.           33          12          24
   71.           34          15          14
   72.           35          14          22
   73.           36          22          19
   74.           37          19          21
   75.           38          21          18
   76.           39          18           1
   77.           40           1           3
   78.           41           3           5
   79.           42           5           6
   80.           43           6           9
   81.           44           9          11
   82.           45          11          24
   83. UNIT INCHES KIP
   84. MEMBER PROPERTY AMERICAN
   85. * CERTAIN TRUSS MEMBERS ARE DOUBLED OR TRIPLED.  THESE MEMBERS
   86. * HAVE ZD = 3 OR 4.5 ACCORDINGLY.
   87. 13 23 TO 33 36 TO 43 PRI YD 9. ZD 3.
   88. 34 35 44 45 PRI YD 9. ZD 4.5
   89. 3 6 8 14 16 19 PRI YD 5.5 ZD 3.
   90. 1 2 4 5 7 9 TO 12 15 17 18 20 TO 22 PRI YD 5.5 ZD 1.5
   91. MEMBER RELEASE
   92. 13 34 36 40 44 START FZ
   93. 33 35 39 43 45 END FZ
   94. MEMBER TRUSS
   95. 1 TO 12 14 TO 22
   96. CONSTANT
   97. E 1700. ALL
  MANZANAR AUDITORIUM - TRUSS                                 -- PAGE NO.    3
 *
   98. SUPPORT
   99. 15 PINNED
  100. 24 FIXED BUT FX FZ MX MY MZ
  101. UNIT FEET POUND
  102. LOAD 1 DEAD + LIVE
  103. *ROOF DEAD
  104. MEMBER LOAD
  105. 34 TO 45 UNI GY -160.
  106. *ATTIC DEAD
  107. MEMBER LOAD
  108. 13 23 TO 33 UNI GY -160.
  109. *ROOF LIVE
  110. MEMBER LOAD
  111. 34 TO 45 UNI GY -320.
  112. *ATTIC LIVE
  113. MEMBER LOAD
  114. 13 23 TO 33 UNI GY -160.
  115. PERFORM ANALYSIS


         P R O B L E M   S T A T I S T I C S
         -----------------------------------

   NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS =    24/   45/    2
   ORIGINAL/FINAL BAND-WIDTH =   17/    3
   TOTAL PRIMARY LOAD CASES =    1, TOTAL DEGREES OF FREEDOM =    69
   SIZE OF STIFFNESS MATRIX =     828 DOUBLE PREC. WORDS
   REQRD/AVAIL. DISK SPACE =  12.07/ 6651.8 MB,  EXMEM = 1958.4 MB


 ++ Processing Element Stiffness Matrix.             7:46:26
 ++ Processing Global Stiffness Matrix.              7:46:26
 ++ Processing Triangular Factorization.             7:46:26
 ++ Calculating Joint Displacements.                 7:46:26
 ++ Calculating Member  Forces.                      7:46:26


  116. UNIT INCHES POUND
  117. PRINT MEMBER FORCES ALL
  MANZANAR AUDITORIUM - TRUSS                                 -- PAGE NO.    4
 *


  MEMBER END FORCES    STRUCTURE TYPE = PLANE
  -----------------

                                                Page 2
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                                         truss.anl
 ALL UNITS ARE -- POUN INCH

MEMBER  LOAD  JT       AXIAL   SHEAR-Y  SHEAR-Z   TORSION      MOM-Y       MOM-Z



    1     1     1   -2437.54       .00      .00       .00        .00         .00
                2    2437.54       .00      .00       .00        .00         .00

    2     1     3    2661.90       .00      .00       .00        .00         .00
                4   -2661.90       .00      .00       .00        .00         .00

    3     1     1   -9844.42       .00      .00       .00        .00         .00
                4    9844.42       .00      .00       .00        .00         .00

    4     1     5    4192.54       .00      .00       .00        .00         .00
                4   -4192.54       .00      .00       .00        .00         .00

    5     1     6    4902.44       .00      .00       .00        .00         .00
                7   -4902.44       .00      .00       .00        .00         .00

    6     1     6   -3480.88       .00      .00       .00        .00         .00
                8    3480.88       .00      .00       .00        .00         .00

    7     1     8     196.70       .00      .00       .00        .00         .00
                5    -196.70       .00      .00       .00        .00         .00

    8     1     9  -10701.92       .00      .00       .00        .00         .00
                7   10701.92       .00      .00       .00        .00         .00

    9     1    10   -4197.83       .00      .00       .00        .00         .00
                9    4197.83       .00      .00       .00        .00         .00

   10     1    10    2252.76       .00      .00       .00        .00         .00
               11   -2252.76       .00      .00       .00        .00         .00

   11     1    11    -456.32       .00      .00       .00        .00         .00
               12     456.32       .00      .00       .00        .00         .00

   12     1    13    2252.71       .00      .00       .00        .00         .00
               14   -2252.71       .00      .00       .00        .00         .00

   13     1    15  -78182.43   1862.11      .00       .00        .00     1641.57
               16   78182.43    279.65      .00       .00        .00    61906.51

   14     1     1   -9845.00       .00      .00       .00        .00         .00
               17    9845.00       .00      .00       .00        .00         .00

   15     1    18    2663.09       .00      .00       .00        .00         .00
               17   -2663.09       .00      .00       .00        .00         .00
   MANZANAR AUDITORIUM - TRUSS                                -- PAGE NO.     5
 *


MEMBER END FORCES    STRUCTURE TYPE = PLANE
-----------------
ALL UNITS ARE -- POUN INCH

MEMBER  LOAD  JT       AXIAL   SHEAR-Y  SHEAR-Z   TORSION      MOM-Y       MOM-Z



   16     1    19   -3480.90       .00      .00       .00        .00         .00
               20    3480.90       .00      .00       .00        .00         .00

   17     1    21    4192.70       .00      .00       .00        .00         .00
               17    4192.70       .00      .00       .00        .00         .00

   18     1    20     196.62       .00      .00       .00        .00         .00
               21    -196.62       .00      .00       .00        .00         .00

   19     1    22  -10701.85       .00      .00       .00        .00         .00
               23   10701.85       .00      .00       .00        .00         .00

   20     1    19    4902.44       .00      .00       .00        .00         .00
               23   -4902.44       .00      .00       .00        .00         .00

   21     1    13   -4197.88       .00      .00       .00        .00         .00
               22    4197.88       .00      .00       .00        .00         .00

   22     1    14    -456.25       .00      .00       .00        .00         .00
               16     456.25       .00      .00       .00        .00         .00
                                         Page 3
```

← Top chord-to-bottom chord connection force : 78 2^k

```
                                                  truss.anl

   23     1      16 -78182.44     176.60       .00        .00        .00  -61906.52
                 13  78182.44    2105.96       .00        .00        .00  -20666.09

   24     1      13 -76046.79    1375.11        00        .00        .00   20666.00
                 23  76046.79     642.17       .00        .00        .00    7056.39

   25     1      23 -84466.55    1061.33       .00        .00        .00   -7056.60
                 20  84466.55    1091.95       .00        .00        .00    5820.51

   26     1      20 -87109.95     976.21       .00        .00        .00   -5820.68
                 17  87109.95    1115.95       .00        .00        .00     338.81

   27     1      17 -78017.21     920.75       .00        .00        .00    -338.93
                  2  78017.21    1218.77       .00        .00        .00  -11616.20

   28     1       2 -78017.20    1218.87       .00        .00        .00   11616.48
                  4  78017.20     920.97       .00        .00        .00     335.79

   29     1       4 -87109.96    1115.99       .00        .00        .00    -335.69
                  8  87109.96     976.17       .00        .00        .00    5821.07

   30     1       8 -84466.52    1091.94       .00        .00        .00   -5821.12
                  7  84466.52    1061.34       .00        .00        .00    7056.50

   31     1       7 -76046.77     642.17       .00        .00        .00   -7056.47
                 10  76046.77    1375.11       .00        .00        .00  -20666.11
  MANZANAR AUDITORIUM - TRUSS                                   -- PAGE NO.     6
 .


 MEMBER END FORCES     STRUCTURE TYPE = PLANE
 -----------------
 ALL UNITS ARE -- POUN INCH

 MEMBER  LOAD  JT      AXIAL    SHEAR-Y  SHEAR-Z    TORSION      MOM-Y       MOM-Z


   32     1      10 -78182.52    2105.96       .00        .00        .00   20666.13
                 12  78182.52     176.60       .00        .00        .00   61906.59

   33     1      12 -78182.43     279.65       .00        .00        .00  -61906.69
                 24  78182.43    1862.11       .00        .00        .00   -1641 62

   34     1      15  83962.00    2596.45       .00        .00        .00   -1641.56
                 14 -82812.88     616.19       .00        .00        .00   86098.98

   35     1      14  80889.77     342.16       .00        .00        .00  -86099.11
                 22 -79664.80    3081.68       .00        .00        .00  -38424.86

   36     1      22  85454.35    2139.10       .00        .00        .00   38424.77
                 19 -85061.22     886.82       .00        .00        .00    9339.44

   37     1      19  88022.36    1391.13       .00        .00        .00   -9339.69
                 21 -87602.83    1838.79       .00        .00        .00   -8885.69

   38     1      21  85095.45    1699.98       .00        .00        .00    8885.62
                 18 -84687.94    1438.26       .00        .00        .00    1467.83

   39     1      18  85030.78    1205.42       .00        .00        .00   -1467.71
                  1 -84614.14    2003.86       .00        .00        .00  -30831.29

   40     1       1  84614.16    2003.96       .00        .00        .00   30831.38
                  3 -85030.80    1205.80       .00        .00        .00    1460.71

   41     1       3  84688.05    1438.37       .00        .00        .00   -1460.59
                  5 -85095.57    1699.87       .00        .00        .00   -8883.39

   42     1       5  87602.80    1838.75       .00        .00        .00    8883.25
                  6 -88022.32    1391.17       .00        .00        .00    9338.94

   43     1       6  85061.20     886.83       .00        .00        .00   -9338.93
                  9 -85454.34    2139.09       .00        .00        .00  -38424.79

   44     1       9  79664.85    3081.68       .00        .00        .00   38424.72
                 11 -80889.81     342.16       .00        .00        .00   86099.10

   45     1      11  82812.88     616.19       .00        .00        .00  -86099.05
                 24 -83962.00    2596.45       .00        .00        .00    1641.61
  MANZANAR AUDITORIUM - TRUSS                                   -- PAGE NO.     7
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                                     truss.anl

  ************** END OF LATEST ANALYSIS RESULT **************

   118. PRINT MEMBER STRESSES ALL
      MANZANAR AUDITORIUM - TRUSS                          -- PAGE NO.    8


  MEMBER STRESSES
  ---------------
  ALL UNITS ARE POUN/SQ INCH

 MEMB   LD  SECT     AXIAL     BEND-Y      BEND-Z    COMBINED  SHEAR-Y  SHEAR-Z


     1     1   .0     295.5 T      .0          .0      295.5       .0       .0
             1.00     295.5 T      .0          .0      295.5       .0       .0

     2     1   .0     322.7 C      .0          .0      322.7       .0       .0
             1.00     322.7 C      .0          .0      322.7       .0       .0

     3     1   .0     596.6 T      .0          .0      596.6       .0       .0
             1.00     596.6 T      .0          .0      596.6       .0       .0

     4     1   .0     508.2 C      .0          .0      508.2       .0       .0
             1.00     508.2 C      .0          .0      508.2       .0       .0

     5     1   .0     594.2 C      .0          .0      594.2       .0       .0
             1.00     594.2 C      .0          .0      594.2       .0       .0

     6     1   .0     211.0 T      .0          .0      211.0       .0       .0
             1.00     211.0 T      .0          .0      211.0       .0       .0

     7     1   .0      23.8 C      .0          .0       23.8       .0       .0
             1.00      23.8 C      .0          .0       23.8       .0       .0

     8     1   .0     648.6 T      .0          .0      648.6       .0       .0
             1.00     648.6 T      .0          .0      648.6       .0       .0

     9     1   .0     508.8 T      .0          .0      508.8       .0       .0
             1.00     508.8 T      .0          .0      508.8       .0       .0

    10     1   .0     273.1 C      .0          .0      273.1       .0       .0
             1.00     273.1 C      .0          .0      273.1       .0       .0

    11     1   .0      55.3 T      .0          .0       55.3       .0       .0
             1.00      55.3 T      .0          .0       55.3       .0       .0

    12     1   .0     273.1 C      .0          .0      273.1       .0       .0
             1.00     273.1 C      .0          .0      273.1       .0       .0

    13     1   .0    2895.6 T      .0        40.5     2936.2     69.0       .0   ← Bottom chord stress
             1.00    2895.6 T      .0      1528.6     4424.2     10.4       .0

    14     1   .0     596.7 T      .0          .0      596.7       .0       .0
             1.00     596.7 T      .0          .0      596.7       .0       .0

    15     1   .0     322.8 C      .0          .0      322.8       .0       .0
             1.00     322.8 C      .0          .0      322.8       .0       .0

    16     1   .0     211.0 T      .0          .0      211.0       .0       .0
             1.00     211.0 T      .0          .0      211.0       .0       .0

    17     1   .0     508.2 C      .0          .0      508.2       .0       .0
             1.00     508.2 C      .0          .0      508.2       .0       .0
      MANZANAR AUDITORIUM - TRUSS                          -- PAGE NO.    9


 MEMBER STRESSES
 ---------------
 ALL UNITS ARE POUN/SQ INCH

MEMB   LD  SECT     AXIAL     BEND-Y      BEND-Z    COMBINED  SHEAR-Y  SHEAR-Z


   18     1   .0      23.8 C      .0          .0       23.8       .0       .0
                                         Page 5
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                                         truss.anl
           1.00    23.8 C       .0          .0      23.8        .0        .0

   19   1   .0    648.6 T       .0          .0     648.6        .0        .0
           1.00   648.6 T       .0          .0     648.6        .0        .0

   20   1   .0    594.2 C       .0          .0     594.2        .0        .0
           1.00   594.2 C       .0          .0     594.2        .0        .0

   21   1   .0    508.8 T       .0          .0     508.8        .0        .0
           1.00   508.8 T       .0          .0     508.8        .0        .0

   22   1   .0     55.3 T       .0          .0      55.3        .0        .0
           1.00    55.3 T       .0          .0      55.3        .0        .0

   23   1   .0   2895.6 T       .0      1528.6    4424.2       6.5        .0
           1.00  2895.6 T       .0       510.3    3405.9      78.0        .0

   24   1   .0   2816.5 T       .0       510.3    3326.8      50.9        .0
           1.00  2816.5 T       .0       174.2    2990.8      23.8        .0

   25   1   .0   3128.4 T       .0       174.2    3302.6      39.3        .0
           1.00  3128.4 T       .0       143.7    3272.1      40.4        .0

   26   1   .0   3226.3 T       .0       143.7    3370.0      36.2        .0
           1.00  3226.3 T       .0         8.4    3234.7      41.3        .0

   27   1   .0   2889.5 T       .0         8.4    2897.9      34.1        .0
           1.00  2889.5 T       .0       286.8    3176.3      45.1        .0

   28   1   .0   2889.5 T       .0       286.8    3176.4      45.1        .0
           1.00  2889.5 T       .0         8.3    2897.8      34.1        .0

   29   1   .0   3226.3 T       .0         8.3    3234.6      41.3        .0
           1.00  3226.3 T       .0       143.7    3370.0      36.2        .0

   30   1   .0   3128.4 T       .0       143.7    3272.1      40.4        .0
           1.00  3128.4 T       .0       174.2    3302.6      39.3        .0

   31   1   .0   2816.5 T       .0       174.2    2990.8      23.8        .0
           1.00  2816.5 T       .0       510.3    3326.8      50.9        .0

   32   1   .0   2895.6 T       .0       510.3    3405.9      78.0        .0
           1.00  2895.6 T       .0      1528.6    4424.2       6.5        .0

   33   1   .0   2895.6 T       .0      1528.6    4424.2      10.4        .0
           1.00  2895.6 T       .0        40.5    2936.2      69.0        .0

   34   1   .0   2073.1 C       .0        27.0    2100.2      64.1        .0  <- Top chord stress
           1.00  2044.8 C       .0      1417.3    3462.0      15.2        .0
 MANZANAR AUDITORIUM - TRUSS                                -- PAGE NO.  10


MEMBER STRESSES
---------------
ALL UNITS ARE POUN/SQ INCH

MEMB   LD  SECT     AXIAL    BEND-Y     BEND-Z  COMBINED  SHEAR-Y  SHEAR-Z


   35   1   .0   1997.3 C       .0      1417.3    3414.5       8.4        .0
           1.00  1967.0 C       .0       632.5    2599.5      76.1        .0

   36   1   .0   3165.0 C       .0       948.8    4113.7      79.2        .0
           1.00  3150.4 C       .0       230.6    3381.0      32.8        .0

   37   1   .0   3260.1 C       .0       230.6    3490.7      51.5        .0
           1.00  3244.5 C       .0       219.4    3463.9      68.1        .0

   38   1   .0   3151.7 C       .0       219.4    3371.1      63.0        .0
           1.00  3136.6 C       .0        36.2    3172.8      53.3        .0

   39   1   .0   3149.3 C       .0        36.2    3185.5      44.6        .0
           1.00  3133.9 C       .0       761.3    3895.1      74.2        .0

   40   1   .0   3133.9 C       .0       761.3    3895.1      74.2        .0
           1.00  3149.3 C       .0        36.1    3185.4      44.7        .0

   41   1   .0   3136.6 C       .0        36.1    3172.7      53.3        .0
           1.00  3151.7 C       .0       219.3    3371.0      63.0        .0

                                         Page 6
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                                        truss.anl
   42     1    .0    3244.5 C        .0      219.3    3463.9      68.1        .0
             1.00    3260.1 C        .0      230.6    3490.7      51.5        .0

   43     1    .0    3150.4 C        .0      230.6    3381.0      32.8        .0
             1.00    3165.0 C        .0      948.8    4113.7      79.2        .0

   44     1    .0    1967.0 C        .0      632.5    2599.5      76.1        .0
             1.00    1997.3 C        .0     1417.3    3414.5       8.4        .0

   45     1    .0    2044.8 C        .0     1417.3    3462.0      15.2        .0
             1.00    2073.1 C        .0       27.0    2100.2      56.1        .0

 ************** END OF LATEST ANALYSIS RESULT **************

 119. PLOT BENDING FILE
 120. PLOT DISPLACEMENT FILE
 121. PRINT SUPPORT REACTIONS
   MANZANAR AUDITORIUM - TRUSS                                -- PAGE NO.   11
  *


 SUPPORT REACTIONS -UNIT POUN INCH    STRUCTURE TYPE = PLANE
 -----------------

JOINT  LOAD   FORCE-X   FORCE-Y   FORCE-Z     MOM-X     MOM-Y     MOM Z


   15    1        .01  32584.51       .00       .00       .00       .00
   24    1        .00  32584.49       .00       .00       .00       .00

 ************** END OF LATEST ANALYSIS RESULT **************

 122. FINISH

          *************** END OF STAAD-III ***************

            **** DATE= SEP 28,1998   TIME=  7.46:27 ****

       *****************************************************
       *        For questions on  STAAD-III,   contact:    *
       *              Research Engineers, Inc at           *
       *  West Coast: Ph- (714) 974-2500  Fax- (714) 921-2543  *
       *  East Coast: Ph- (508) 688-3626  Fax- (508) 685-7230  *
       *****************************************************
```

$$\frac{34 \text{ plf}}{2'} = 17 \text{ psf}$$

$$\ell/240 = (16)(12)/240 = 0.8$$

$$\Delta = \frac{5w\ell^4}{384EI} \rightarrow 0.8 = \frac{(5)(w)(16)^4(1728)}{(384)(1700000)(1.5)\frac{(5.375)^3}{12}} \rightarrow w = 18 \text{ psf} \quad (D+L)$$

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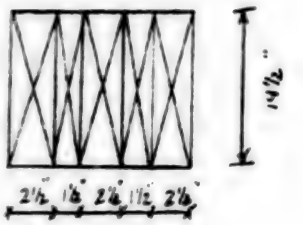
Park MANZ (DEVA 004-35)	**NATIONAL PARK SERVICE** **DENVER SERVICE CENTER**		**Sheet** 13
Area AUDITORIUM			**of**
Project	**By** E. SMITH	**Checked**	**Pkg.**
Feature	**Date** 9-24-98	**Date**	**Account**

BUILT-UP COLUMN

Assume



2 conditions:

1. $d = 14\tfrac{1}{2}"$, $\ell_c = 15'$, $K_f = 1$
2. $d = 10\tfrac{1}{2}"$, $\ell_c = 11'$, $K_f = 0.75$

① $F_{cE} = \dfrac{(0.3)(1700,000)}{\left[(15)(12)/14.5\right]^2} = 3309$

$F_c^* = F_c \cdot C_D \cdot C_M \cdot C_F = (1450)(1.15)(1)(0.9) = 1500$

$C_p = 0.88$

② $F_{cE} = \dfrac{(0.3)(1700,000)}{\left[(11')(12)/10.5\right]^2} = 3227$

$C_p = 0.66$ (Includes K_f)

$P = F_c' \cdot A = (0.66)(1500)(10.5")(14.5") = 151^k > \overset{\text{From STAAD anal.}}{32.6^k}$

However, bolting pattern does not meet built-up column requirements.

Check to see if column is ok as is. (See next page.)

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Park MANZ (DEVA 004-35)	**NATIONAL PARK SERVICE**		**Sheet** 14
Area AUDITORIUM	**DENVER SERVICE CENTER**		**of**
Project	**By** E. SMITH	**Checked**	**Pkg.**
Feature	**Date** 9-28-98	**Date**	**Account**

BUILT-UP COLUMN, CONT.

CAPACITY OF 1½" x 14½"

2 Conditions:

1. $l_e/d = (15')(12)/14.5" = 12.4$

2. $l_e/d = (5')(12)/1.5" = 40$ ← (Note: 5' is approx. spacing between bolts.)

$$F_{cE} = \frac{(0.3)(1700,000)}{40^2} = 319$$

$F_c^* = 1500$

$C_p = 0.20$

$P = (0.20)(1500)(1.5")(14.5") = 6525^{lb}$

CAPACITY OF 2½" x 14.5"

$$F_{cE} = \frac{(0.3)(1700,000)}{\left[(15')(12)/2.5\right]^2} = 885$$

$C_p = 0.49$

$P = (0.49)(1500)(2.5")(14.5") = 26,640^{lb}$

$\Rightarrow P_{TOTAL} = (3)(26.6^k) + (2)(6.5^k) = 92.8^k > 32.6^k$

$\underline{\underline{OK}}$

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FEMA 178 / June 1992 Seismic Evaluation

MANZANAR AUDITORIUM BUILDING

Manzanar National Historic Site
Inyo County, California

FINAL REPORT

December 1998

Prepared for

The National Park Service
Denver Service Center
12795 W Alameda Parkway
P.O. Box 25287
Denver, Colorado 80255

Prepared by

CONSULTING
MARTIN/MARTIN
ENGINEERS

MARTIN/MARTIN, Inc
4251 Kipling
Wheat Ridge, Colorado 80034

NPS Project No. DSC-605-17A
Task Order No. 3
Contract No. 1443-CX-2000-97-037
Contract Date: July 18, 1997

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MARTIN/MARTIN

Consulting Engineers
4251 Kipling, P.O. Box 4001
Wheat Ridge, Colorado 80034-4001
303 431 6100

December 2, 1998

Mr. Brian Tallent, P.E.
National Park Service
12795 W. Alameda Pkwy.
Denver, CO 80225

RE: FEMA 178 Seismic Evaluation of the Manzanar Auditorium Building
Inyo County, California
MARTIN/MARTIN Project No. 13310.04

Dear Brian,

MARTIN/MARTIN is pleased to submit this final report for your use. A seismic evaluation of the Manzanar Auditorium Building was performed in accordance with FEMA 178 / June 1992 at the request of the National Park Service in accordance with Task Order 3 of Contract No. 1443CX2000-97-037.

The Manzanar Auditorium Building is a wood framed structure constructed in 1944 by the Federal Government for the Manzanar war relocation camp and remodeled in 1954 by Inyo County for use as the county maintenance facility. The building is deteriorated in places on the outside, but is in fair condition inside.

This evaluation uncovered several deficiencies in the seismic load resisting systems. These include:

1. Decay at the bottom of wood columns on the south wall and potential decay of wood below the slab-on-grade on the north, east and west auditorium walls.
2. Lack of adequate shear walls in the auditorium space and the adjoining north, east, and west wings.
3. Inadequate bracing of the north wing cripple walls enclosing the crawl space.
4. Inadequate roof diaphragms due to straight (i.e. perpendicular) 1x roof sheathing on all roofs.
5. Inadequate roof diaphragms due to discontinuous diaphragm chords and collectors on all roofs.

6. Inadequate connections and ties between structural elements including:
    - Wood columns in the auditorium and posts in the crawl space of the north wing do not appear to be connected to the supporting concrete foundations.
    - Spread footing foundations in the north wing are not tied together.
    - Shear walls are not adequately connected to foundations.
    - The roof diaphragms are not adequately connected to the walls.
7. Top chords of the wood trusses are cracked at the knee brace connection locations.
8. The ceiling of the auditorium space lacks horizontal bracing.

Due to these deficiencies, the building is expected to perform poorly in the "design" seismic event. Rehabilitation is recommended. Principal ideas for rehabilitation are to add new concrete foundations to separate wood from soil, create a new roof diaphragm by adding plywood, make the diaphragm chords and collectors continuous by adding light gauge steel connectors, and create new shear walls or steel rod braced frames at selected bays. The foundations in these selected bays must also be enhanced.

The attached report discusses these items in detail.

Sincerely,

Paul Doak, P.E., S.E.
Principal

REGISTERED PROFESSIONAL ENGINEER
PAUL DOAK
S-3669
Exp. 9-30-200
STRUCTURAL ENGINEER
STATE OF CALIFORNIA

PD/tm

Enclosures

G:\13310_04\WP\MANZANAR\MANZ-LTR.DOC

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MARTIN/MARTIN

Consulting Engineers

FEMA 178 Seismic Evaluation
Manzanar Auditorium Building
December, 1998

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FEMA 178 Seismic Evaluation
Manzanar Auditorium Building
December, 1998

APPENDIX A

FEMA 178 Data Summary Sheet A1

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APPENDIX B

Calculations .... B1.1 – B3.2

APPENDIX C

Glossary of Terms C1

MARTIN/MARTIN

Consulting Engineers

FEMA 178 Seismic Evaluation
Manzanar Auditorium Building
December, 1998

1.0 INTRODUCTION

At the request of the National Park Service, MARTIN/MARTIN conducted a seismic evaluation of the Manzanar Auditorium Building in Inyo County, California. The work conforms to FEMA 178/June 1992 as modified in the Task Order #3 contract. These limitations are described in Section 5.2.

The evaluation consisted of a data gathering phase conducted onsite July 20, 21, and 22, 1998; an analysis phase in which base shears and stress checks were performed; and development of ideas for rehabilitation.

This seismic evaluation was performed on the building as it existed during our site visit in July 1998 Figure 1 contains a ground floor plan with the different building areas labeled. This figure, as well as all others, is from the Historic American Building Survey conducted in 1994 by the National Park Service.

2.0 BUILDING DESCRIPTION

The Manzanar Auditorium Building was constructed in 1944 by the United States Government as an assembly building for the Manzanar War Relocation Center. The construction is wood framing supported by concrete slab-on-grade and concrete spread footings. In 1954, the building ownership was transferred to Inyo County, and it then served as a county maintenance facility. Numerous modifications to the structure were made by the county including removing the stage area in the east wing, replacing the raised wood floor in the auditorium with concrete slab-on-grade, and adding a large wall opening to the east exterior wall (Photo Nos. 1 and 2) and the south exterior wall near the southwest corner (Photo Nos. 3 and 4). The south wing of the building was removed in the early 1950's.

2.1 GRAVITY LOAD RESISTING SYSTEM

Auditorium:

The roof of the auditorium consists of 1x wood sheathing on 2x10 joists spaced at 24 inches on center, spanning to wood trusses spaced at 16 feet on-center (Photo No. 7). The 1x sheathing is placed straight (i.e. perpendicular) to the joists. The trusses span approximately 80 feet and are supported by built-up wood columns along the north and south walls of the auditorium. The trusses and columns are knee braced (Photo No. 5 and Figure No. 8). The ceiling consists of Celotex panels supported by 2x6 ceiling joists spaced at 24 inches on center spanning to the truss bottom chord (Photo No. 8).

Exterior building walls consist of horizontal lapped siding (vertical siding at the west elevation) over diagonal 1x redwood sheathing attached to 2x4 or 2x6 studs spaced at 16 to 24 inches on center (Photo Nos. 9, 10, and 11). Walls, which were exterior in the original construction, have diagonal redwood sheathing while the interior walls have horizontal 1x sheathing (Photo No.

12). The interior wall covering is horizontal 1x lapped siding (Photo No. 6). Several walls which are now exterior were originally interior. These do not have diagonal redwood sheathing.

There are large areas of windows in the auditorium north and south walls and a large sliding door opening in the south wall near the west corner. The east wall of the auditorium was originally the proscenium wall for the stage. A proscenium truss remains spanning 29 feet to wood columns. The original 29 foot stage opening has been partially infilled; a 14 foot wide opening remains for an overhead coiling door.

The wood columns are supported on tapered concrete footings. As mentioned, the floor of the auditorium is concrete slab-on-grade. The west wall of the auditorium is supported by 28" x 28" concrete spread footings spaced approximately 8 feet on center. The top of footing elevation is about 16 inches below interior slab-on-grade elevation. The exterior west wall is assumed to be supported on wood beams and posts in the area between the floor and footings, but this was not visible and therefore not confirmed.

The auditorium north wall is carried on wood beams supported on wood posts set on concrete spread footings (Photo No. 16).

The auditorium south wall has diagonal 1x sheathing from the roof down to the window sill elevation. Below this, only horizontal sheathing exists because this was an interior wall in the original construction when the south wing was present. The original support for the south wall is assumed to be wood beams and posts to match the north wall. These have been removed and a concrete grade beam added to close off and support the bottom of wall. Exterior grade was also raised which placed the bottom of wood columns below finished grade (Photo No. 14).

The auditorium east wall, either side of the stage area, has horizontal 1x sheathing on 2x4 studs extending down to the furnace room and storage room grade beams. This wood is also below the auditorium slab-on-grade elevation and therefore the west wall face appears to be exposed to soil. This wall sandwiched a remaining section of raised wood floor in the auditorium (Photo No. 21). This floor framing remnant now reduces the load carrying capacity of the wall because the wall lacks horizontal bracing at this level.

North Wing:

The north wing framing consists of straight 1x roof sheathing on 2x12 wood joists spaced at 24 inches on center which span between the auditorium north wall and the north wall of the north wing. These load-bearing walls consist of 2x4 studs spaced at 24 inches in center. The few interior walls in the north wing are constructed with 2x4 studs with horizontal 1x sheathing. The wood floor of the north wing covers a 3 foot high crawl space. The wood floor consists of wood flooring applied over diagonal 1x wood subfloor supported by 2x6 floor joists spaced at 12 or 16 inches on center, which are carried by wood beams spanning between wood posts resting on concrete footings (Photo No. 15). The north wing has a perimeter cripple wall which encloses the crawl space (Photo Nos. 15 and 17). The footings supporting the perimeter posts of the north wing are exposed on the exterior (Photo No. 9).

East Wing:

The east wing has been extensively revised since the original construction. A large 14 foot wide door opening was added on the east elevation, and a 30 foot wide section of the stage was removed. This is currently the main entry into the building and goes under the existing proscenium truss.

The roof framing in this area is straight 1x roof sheathing on 2x12 joists spanning east-west spaced at 24 inches on center and supported by the load bearing exterior east wall of 2x4 and 2x6 studs and the load bearing auditorium east wall. Foundations under the east exterior wall consist of a concrete grade beam, which was visible in the low storage rooms beneath the original stage. Exterior grade on the east side is above the top of grade beam; the wood wall framing is in contact with soil.

Floors in the east wing consist of wood flooring over diagonal 1x wood subfloor on 2x joists. Joists are spaced at 12 inches on center in the remaining stage area.

West Wing:

The west wing consists of the projection room and original main entry. The roof construction is straight 1x sheathing over 2x8 joists spaced at 24 inches on center which are supported by load bearing 2x4 studs. The floor of the projection room has 2 inches of concrete over wood subflooring, and is supported on 2x4 wall studs. The original ground floor in the west wing appears to be concrete slab-on-grade and the walls appear to be supported by a down-turned edge of the slab-on-grade.

2.2 LATERAL LOAD RESISTING SYSTEM

Auditorium:

In the north-south direction, the auditorium has two lateral load resisting systems. The knee-braced trusses can act as moment frames, and the wood walls on grids 2 and 8 can act as shear walls. In the east-west direction, the only apparent lateral system is the roof diaphragm spanning between the north and south auditorium walls (on grids A and H), which must act as shear walls

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North, East, and West Wings:

The north, east and west wings have a lateral system consisting of 1x roof and floor sheathed diaphragms spanning to diagonal 1x sheathed walls which act as shear walls. Some diagonal blocking was observed in the interior, horizontally sheathed walls of the east wing (Photo No. 12). These are helpful but have limited strength and stiffness.

The building characteristics are summarized on the FEMA 178 Data Summary Sheet in Appendix A.

3.0 DEFICIENCIES IN THE SEISMIC LOAD RESISTING SYSTEM

3.1 LIST OF DEFICIENCIES

The primary deficiencies are summarized below. The deficiencies are noted in the FEMA 178 evaluation statements contained in Appendix A. The deficiencies are ranked in order of importance with the first item judged as being the most detrimental.

3.1.1. Wood Decay

The wood columns on the south elevation are decayed. All five built-up wood columns on the south wall of the building are in contact with soil. Three show signs of decay caused apparently by termite damage. The worst column has about 12 inches completely removed out of a total column depth of 15 inches (Photo Nos. 13 and 14). The other two columns are expected to be decayed as well, but there is no visible damage.

The decayed columns have reduced strength to support gravity loads, and reduce the strength of the truss-kneebrace-column lateral system.

Wood on the north, west and east auditorium walls also extends below the interior slab-on-grade elevation by about 16 inches and is apparently in contact with soil. Therefore, decay is also expected in these bearing walls.

3.1.2. Inadequate Shear Walls

Auditorium and North Wing:

There are inadequate shear walls in the east-west direction of the auditorium. The north and south walls have too many wall openings (both windows and doors), diagonal 1x sheathing (and horizontal 1x sheathing below the window sill elevation) instead of plywood sheathing or rod bracing, and inadequate foundations to resist shear and overturning reactions. The north and west walls of the north wing have numerous window and door openings, leaving little wall to act as shear wall. The common wall between the north wing and the auditorium, although having some solid wall sections, has horizontal (i.e. perpendicular) 1x sheathing because it is an interior wall. Perpendicular sheathed walls are inadequate to resist seismic forces because they have little strength and are flexible. They are not accepted by building codes.

East and West Wings:

The north shear wall of the east wing (on grid G) is discontinuous at the ground floor; it rests on the wood floor framing instead of extending down to a foundation.

3.1.3. Unbraced Cripple Walls

The north wing cripple wall surrounding the crawl space exterior is inadequately braced. Some 2x6 diagonal braces were observed but these do not provide the required strength due to their slenderness and poor connections.

3.1.4. Inadequate Roof Diaphragm Sheathing

All roof diaphragms have straight (i.e. perpendicular) 1x roof sheathing instead of diagonal 1x roof sheathing or plywood. Perpendicular sheathed roof diaphragms are also inadequate to resist seismic forces are also not accepted by building codes.

3.1.5. Inadequate Roof Diaphragm Chords and Drag Struts

The double 2x top wall plates cannot function as the roof diaphragm chords or as diaphragm drag struts because the plates are not adequately spliced. At the north and south auditorium walls, the top plates are interrupted at each column (Photo No. 19). In addition, the top plates on the east and west auditorium walls are butt spliced at each break in roof slope (Photo No. 10).

3.1.6. Inadequate Connections and Ties

There are inadequate connections and ties between the various structural components. These are grouped into four categories. One, the auditorium roof diaphragm is not adequately tied to the north and south shear walls on grids A and H.

Two, the shear walls are not connected to the foundation. There are some ½ inch diameter sill bolts but they are too widely spaced to be effective or altogether missing. Due to the aspect ratio (height to length) of the shear walls, wall holddowns to resist wall overturning forces are required, but none were observed. Also, many walls bear on wood beams on wood posts and as such are not connected to the foundation. These walls have little shear and overturning resistance and therefore cannot function as shear walls.

Three, the wood columns do not appear to be connected to their foundation, although this could not be confirmed by visual observation. On the north wing floor framing the floor beams are not connected to the supporting posts, and the posts are not connected to the spread footings (Photo No. 15).

Four, the footing foundations are not interconnected on the north wing (Photo No. 15). Interconnection of footings is necessary to prevent individual footings from moving.

3.1.7. Roof Truss Cracking

The roof truss top chords constructed of built-up 1½ inch and 2½ inch wide members have a noticeable split at the panel point where the knee brace connects to the top chord (Photo No. 20). This is typical at each end of each truss.

3.1.8. Ceiling Bracing

The 2x framing with Celotex panels forming the auditorium ceiling has no horizontal bracing (Photo No. 8). The west end wall, on grid 8, is framed with a joint at the ceiling level (platform framing). The west end wall relies on the ceiling to laterally support the wall, but the ceiling is unable to provide this bracing.

3.2 CONSEQUENCE OF DEFICIENCIES

The building has serious decay in the wood columns and suspected decay in wood bearing walls, lacks adequate diaphragms and shear walls, lacks cripple wall bracing, and lacks connections between structural elements. The building is expected to perform poorly when subject to the design earthquake. Partial or total collapse may occur. The cracked roof trusses and decayed wood columns also reduce the gravity load (dead, live, and snow load) carrying capacity of the building.

These deficiencies need to be corrected to ensure the safety of the building occupants.

4.0 REHABILITATION

4.0.1. Wood Decay

The decay of the wood columns on the south wall can be repaired. The damaged wood can be removed and replaced by installing a new wood column, or preferably, the concrete support can be raised to avoid wood/soil contact. This is shown in Figure 7. It may also be possible to repair the columns having minor decay with epoxy, but decay will continue unless the wood is separated from soil.

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The wood below slab-on-grade elevation at the auditorium north, east, and west walls is more difficult to correct. The first step is to confirm this suspected condition by removing selected portions of the interior slab-on-grade down to the footing elevation. If wood is in contact with soil, a new concrete grade beam could be added between footings. These are shown in Figure 2. Alternately, the concrete slab-on-grade could be removed, interior grade lowered to the footing elevation, and a new wood floor over crawl space installed. This solution has the added benefits of being an historically accurate recreation of the original construction and offers opportunities to route electrical, plumbing, and HVAC services under the floor. This solution is expected to be more costly than adding new concrete grade beams.

4.0.2. Inadequate Shear Walls

The lack of adequate shear walls is a significant problem. The large number of wall openings, both windows and doors, combined with 1x sheathing renders the walls severely overstressed. It is suggested a bay or two on each wall be selected and designed as a new plywood shear wall or new diagonal steel rod braced frame. These must extend from the roof level down to the foundations. These added shear walls and/or bracing would act as the vertical component of the lateral load system for the building; the existing walls should not be combined with the new walls/bracing. Overturning problems are expected at the base of these bays. A new concrete grade beam with new footings or enlarged existing new footings will be required to transfer the lateral shear and overturning forces into the supporting soil. The suggested locations for added shear walls/diagonal braces are shown in Figures 2, 4, 5, 6, and 7.

The added walls or braces on the north wing way may be objectionable if the existing windows are to be kept clear. There are alternate (also more expensive) means, such as moment frames, of providing lateral support. These issues should be explored during the building renovation design.

4.0.3. Unbraced Cripple Walls at North Wing Crawl Space

The added shear walls or diagonal bracing in the north wing can be designed to also brace the crawl space.

4.0.4. Inadequate Roof Diaphragms

The inadequate roof diaphragms caused by perpendicular 1x roof sheathing can be resolved by removing the roofing and installing an engineered plywood roof diaphragm over the 1x sheathing. This is shown in Figure 3. An alternate solution in the auditorium is to locate the roof diaphragm at ceiling level by augmenting the ceiling bracing to also function as the building diaphragm. Vertical bracing must be added between the roof and the ceiling with this solution, but it avoids the cost of re-roofing. A similar idea may be used for the north, east and west wings by attaching the new plywood to the underside of the roof joists.

The roof gravity load carrying capacity must be checked in either case to ensure the added weight can be safely supported.

4.0.5. Lack of Diaphragm Chords and Drag Struts

The two 2x wood top wall plates are prevented from acting as a diaphragm chord because they are discontinuous. This is readily remedied by installing light gauge steel straps across the discontinuity. The steel straps must be engineered and properly nailed.

The added shear walls or diagonal bracing bays in item 4.0.2 are at discrete locations. Diaphragm drag struts are needed to receive the uniform shear from the roof diaphragms and deliver the concentrated lateral force to the added shear wall/braced bay. The two 2x top wall plates may function as the drag struts provided they are adequately sized and connected.

4.0.6. Inadequate Connections and Ties

Inadequate connections and ties between structural components can be solved for each deficient category.

- The roof diaphragm which is being enhanced with plywood, can be connected to the diaphragm chords and collectors at the same time.

- The existing wall connections to the foundations are no longer a concern because new shear walls/rod bracing will be used with new foundations.

- The north wing beam-to-post and post-to-foundation connections can be added using light gauge connectors fastened to the wood and expansion bolted to the concrete.

- The requirement to interconnect foundation elements should be reviewed by a goetechnical engineer who considers the seismic characteristics of the site and the stability of the surface soils. It is probable this requirement can be removed. If it remains, new steel elements (angles or pipes) could be used by expansion bolting them to the footings. Alternatively a 3 to 4 inch thick concrete slab-on-grade added to the ground in the crawl space should adequately interconnect the existing footings if properly designed and doweled to the footings.

4.0.7. Roof Truss Cracking

The cracked roof trusses can be repaired by extending new metal straps from the diagonal truss members around the top of the top chord and back down the opposite side of the diagonal member (Figure 8). This new metal strap should be designed to transfer all of the tension in the diagonal member. This will avoid imposing cross grain tension on the top chord members.

4.0.8. Ceiling Bracing

Replacing the existing Celotex ceiling with a gypsum board ceiling will improve the bracing but still does not satisfy the building code due to the 80 foot ceiling diaphragm span. Alternatively, 2x wood diagonals can be added in the horizontal plane of the ceiling, or plywood added in alternate bays to the top or bottom of the ceiling joists (Figure 3). This solution can also be designed to act as the primary building diaphragm as discussed in item 4.0.4.

Still another solution is to add more vertical bracing from the ceiling to the roof. Currently only one line of vertical bracing occurs at truss midspan and these are flimsy, single 2x wood bracing members. Vertical bracing could be installed at each, or perhaps alternate, truss panel points. This would reduce the span length of the ceiling diaphragm by transferring load up to the roof diaphragm.

5.0 SUMMARY

5.1 CONCLUSIONS

The Manzanar Auditorium Building, according to this evaluation, is vulnerable to collapse during a significant earthquake. However, this conclusion is tempered with the fact that the building has withstood perhaps minor earthquakes and certainly significant winds during its 56 year life. Wood buildings have inherent redundancies and abilities to resist lateral loads that are difficult to quantitatively evaluate. In addition, the main auditorium building is buttressed by the north, east, and west wings, and the quality of workmanship and the quality of timber used 50 years ago is often superior to those used today.

Nonetheless, the building has not experienced a design level earthquake and this evaluation is intended to address that occurrence. It is recommended that the seismic load resisting system be strengthened when the building is remodeled. The required work is relatively straightforward and does not require special construction expertise. It can be relatively economically incorporated during the remodeling period. Many of the rehabilitation suggestions need to be considered concurrent with the remodel. The most economical solution will be one that fits readily into the remodel. The final decision on which solution to employ and its location should not be made until the remodel and seismic rehabilitation work are considered concurrently.

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5.2 REPORT LIMITATIONS

This report did not address the following items at the request of the National Park Service:

1. An evaluation of the gravity load capacity of the structure was not performed.
2. A seismic evaluation of components, cladding and other nonstructural elements was not performed. This is not a significant issue on this building because there are very few nonstructural elements present.
3. Selective demolition was not performed to uncover hidden areas. Therefore, questions such as the possible connection between wood columns and footings and wood exposed to soil could not be addressed.
4. No quantitative material testing was performed.
5. This evaluation was only for the life-safety criteria contained in FEMA 178. The criteria used asks whether the building will remain standing in order to allow occupants to exit safely after an earthquake. Evaluation and rehabilitation recommendations are not offered to address continued operation of the building during or after an earthquake, to address the reduction of repair costs after an earthquake, or to address the reduction of damage to nonstructural elements or building components.
6. The soils and geologic information herein are based on observations of the foundations and the surface soils. The seismic site coefficient was estimated based on these observations. A detailed geotechnical report was not available for the site. Likewise, liquefaction potential and evaluation of surface fault rupture potential are unknown.
7. A construction cost estimate is not included.

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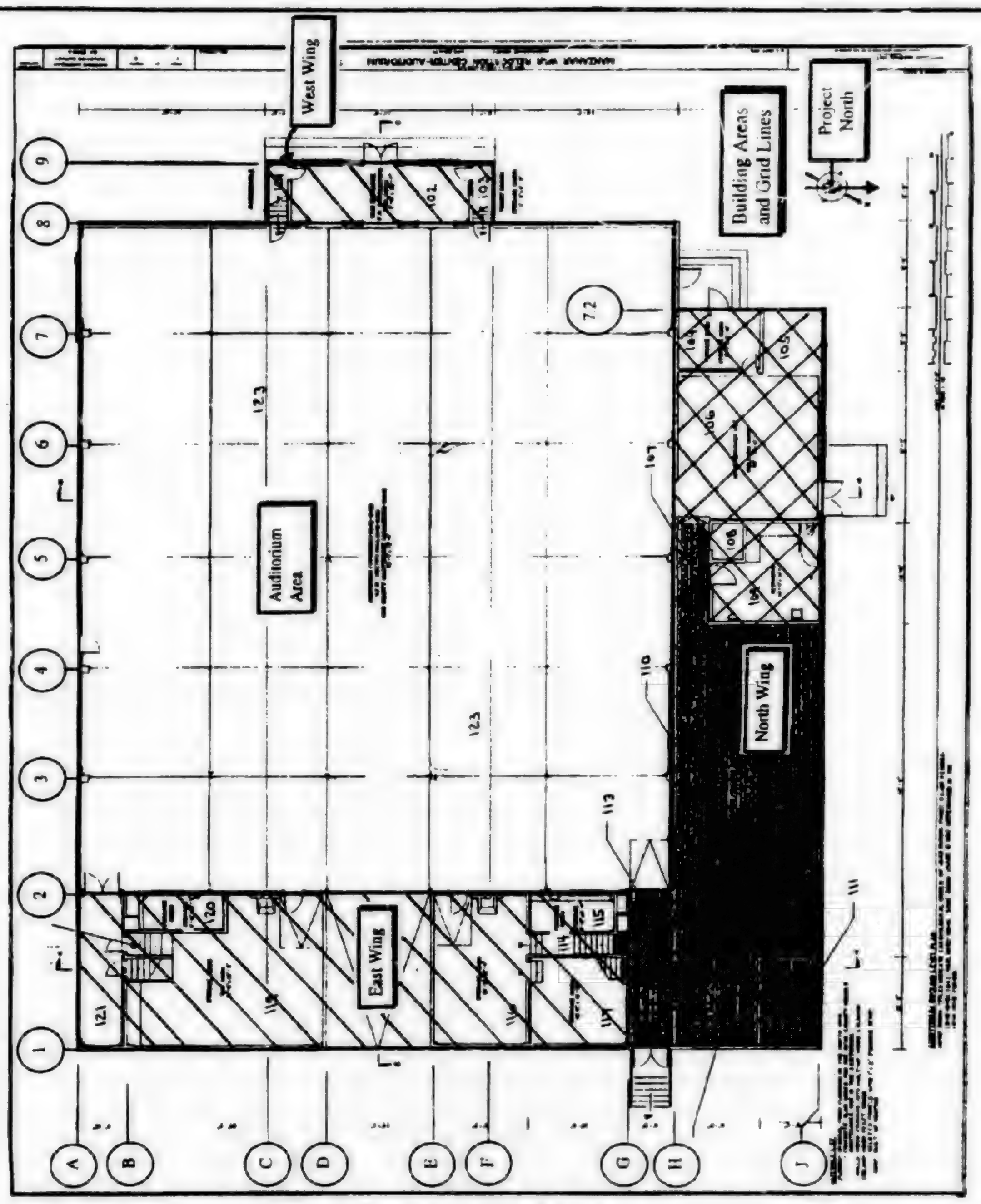

Figure 1 Building Areas

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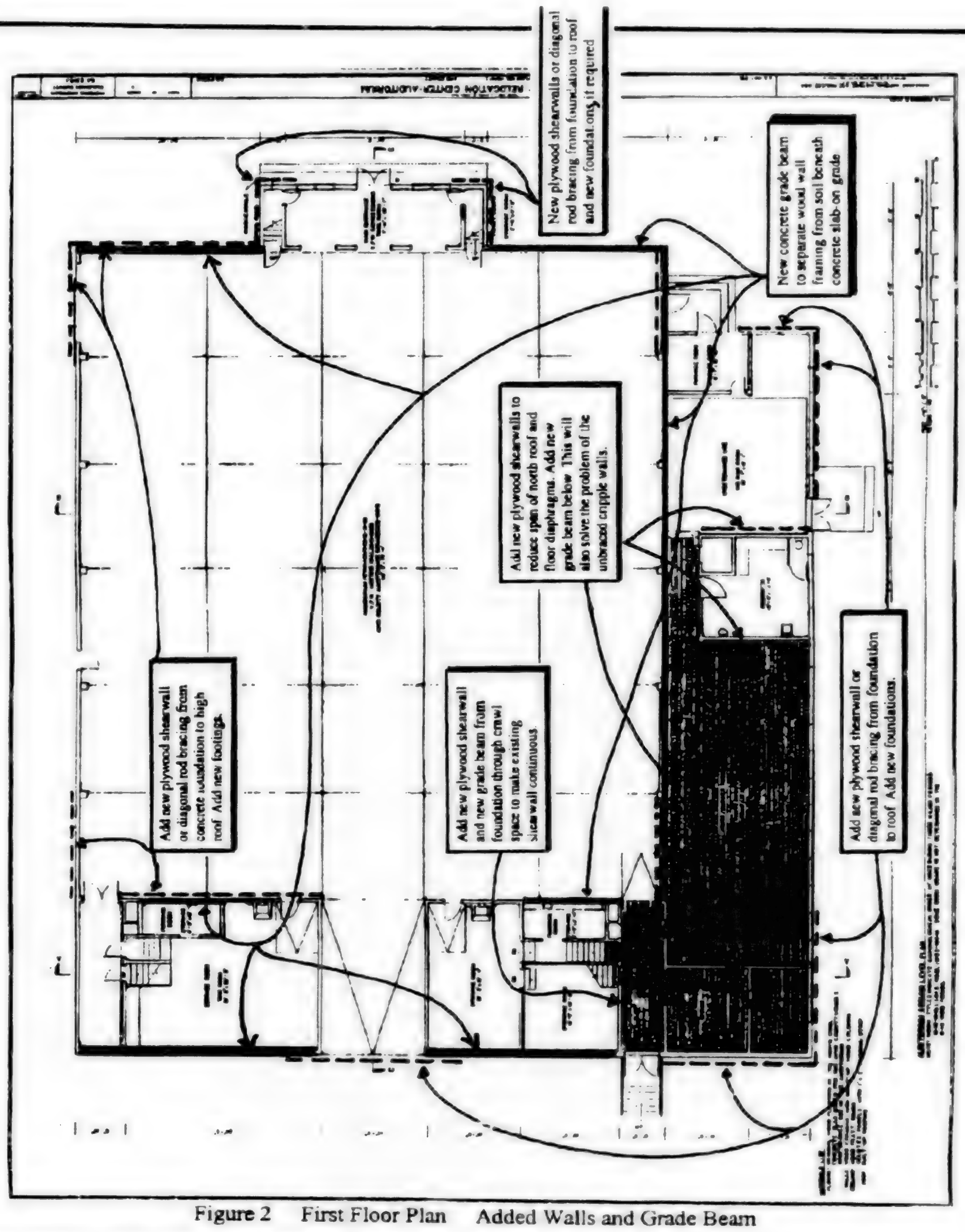

Add new plywood shearwall or diagonal rod bracing from concrete foundation to high roof. Add new footings.
Add new plywood shearwall and new grade beam from foundation through crawl space to make existing shearwall continuous
Add new plywood shearwalls to reduce span of north roof and floor diaphragms. Add new grade beam below. This will also solve the problem of the unbraced cripple walls.
New plywood shearwalls or diagonal rod bracing from foundation to roof and new foundations if required
New concrete grade beam to separate wood wall framing from soil beneath concrete slab-on grade
Add new plywood shearwall or diagonal rod bracing from foundation to roof. Add new foundations.

Figure 2 First Floor Plan Added Walls and Grade Beam

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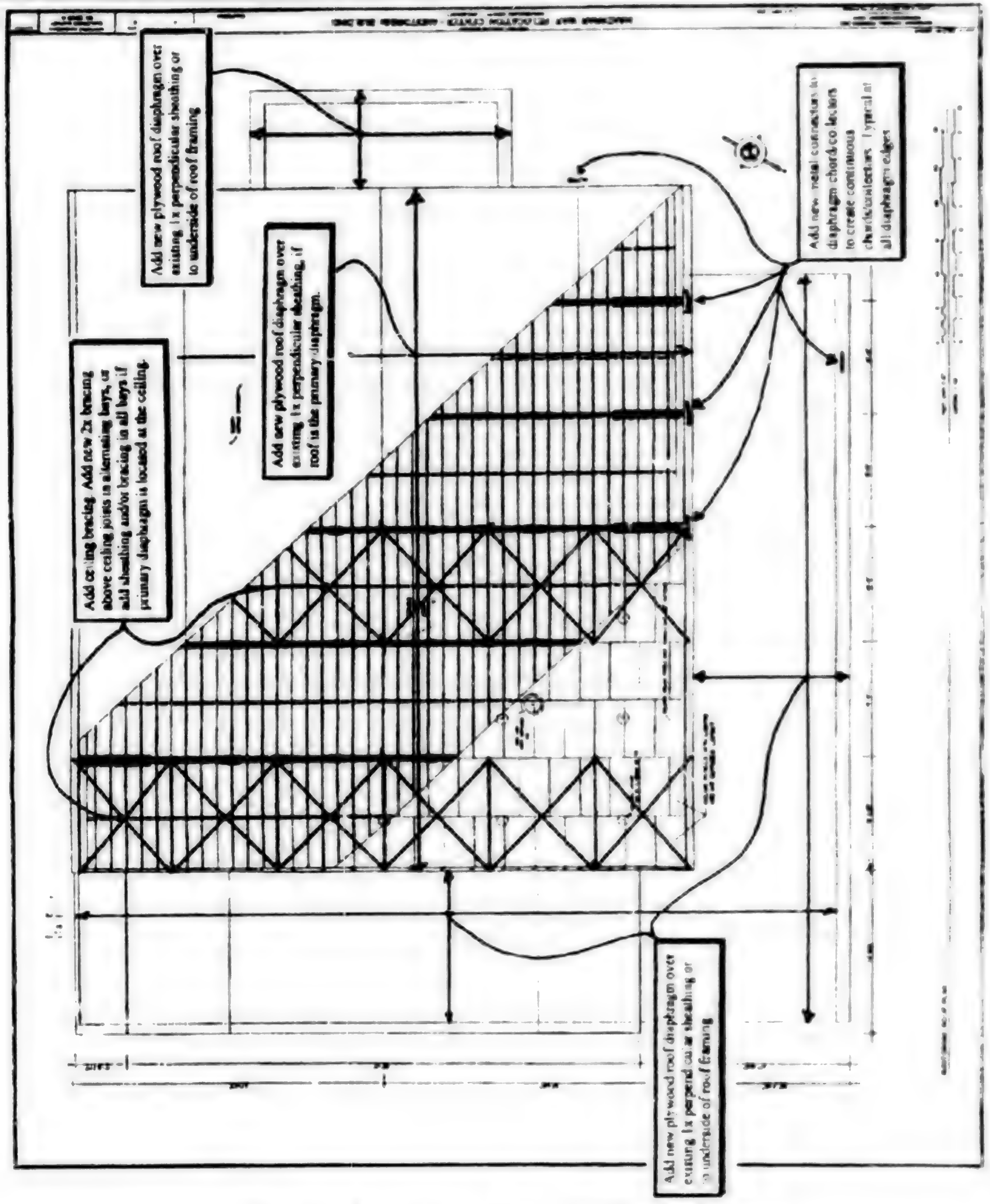
Add ceiling bracing. Add new 2x bracing above ceiling joists in alternating bays, or add sheathing and/or bracing in all bays if primary diaphragm is located at the ceiling.
Add new plywood roof diaphragm over existing 1x perpendicular sheathing or to underside of roof framing
Add new plywood roof diaphragm over existing 1x perpendicular sheathing, if roof is the primary diaphragm.
Add new metal connectors to diaphragm chord/collectors to create continuous chords/collectors. Typical at all diaphragm edges
Add new plywood roof diaphragm over existing 1x perpendicular sheathing or to underside of roof framing

Figure 3 Roof / Ceiling Plan Added Roof Diaphragm

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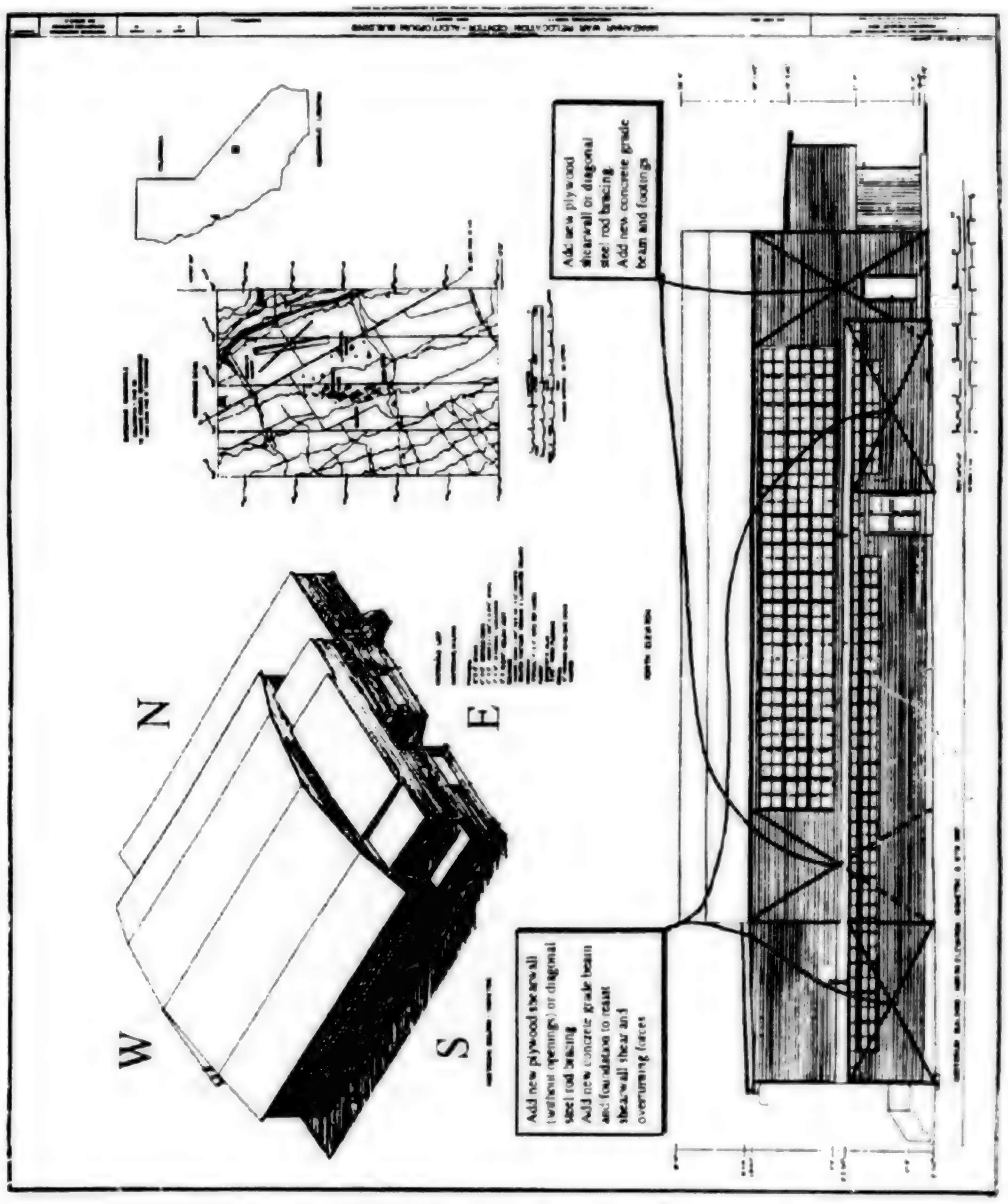
W
N
S
E
Add new plywood shearwall (without openings) or diagonal steel rod bracing
Add new concrete grade beam and foundation to resist shearwall shear and overturning forces
Add new plywood shearwall or diagonal steel rod bracing.
Add new concrete grade beam and footings

Figure 4 North Elevation Added Bracing

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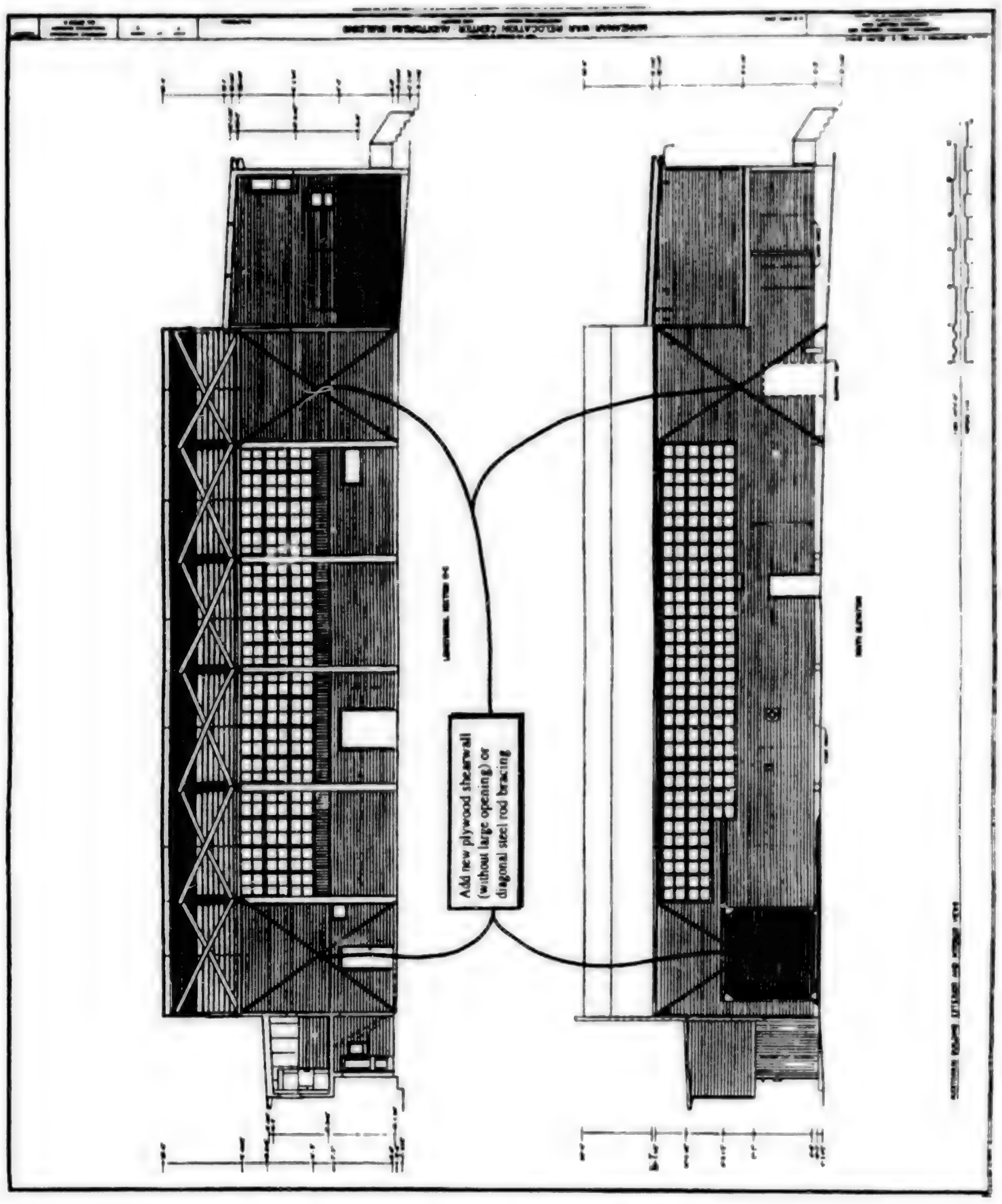
Add new plywood shearwall (without large opening) or diagonal steel rod bracing

Figure 5 Auditorium Area Added Bracing

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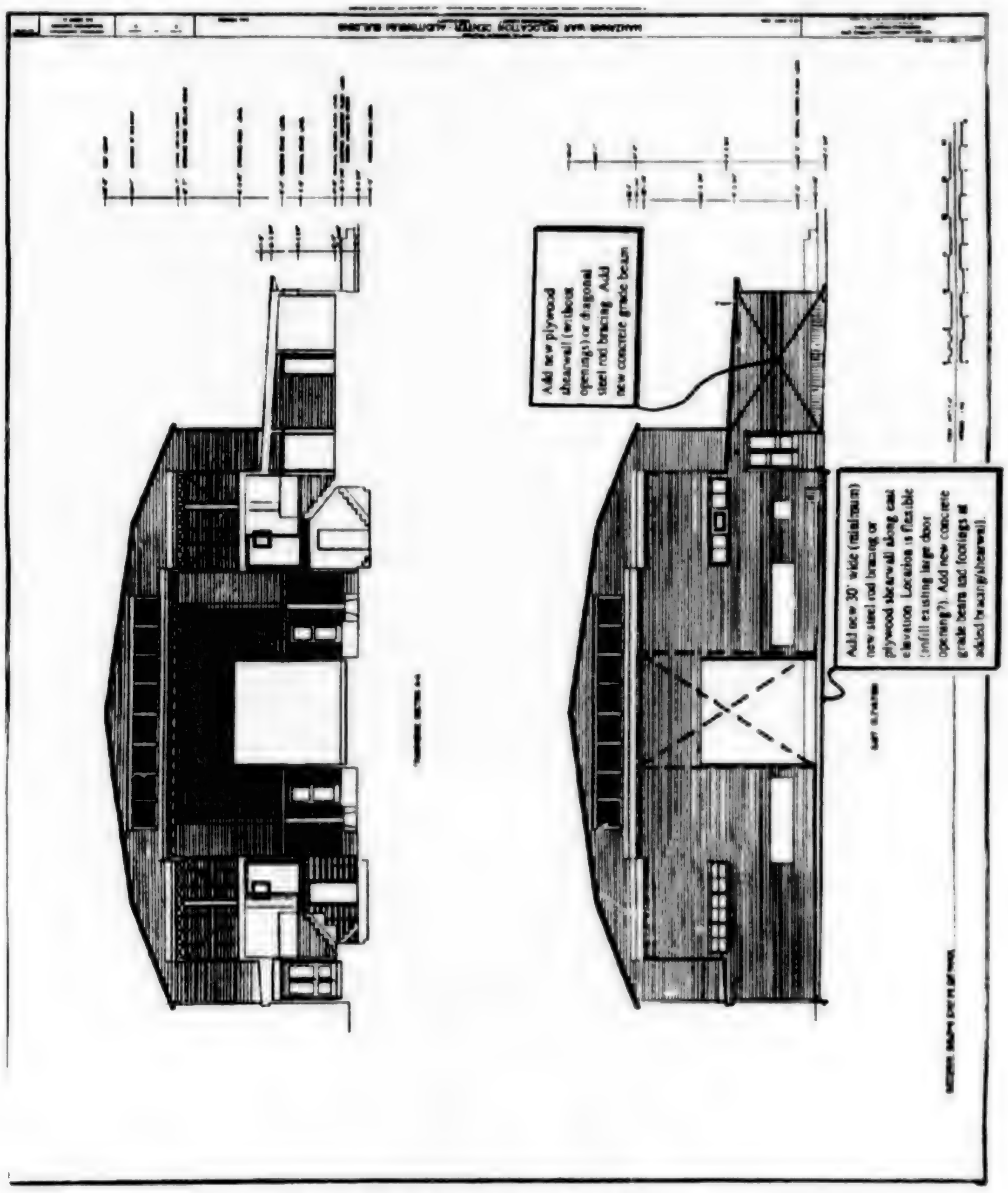

Add new plywood shearwall (without openings) or diagonal steel rod bracing. Add new concrete grade beam

Add new 30' wide (minimum) new steel rod bracing or plywood shearwall along east elevation. Location is flexible (infill existing large door opening?). Add new concrete grade beam and footings at added bracing/shearwall.

Figure 6 East Elevation Added Bracing

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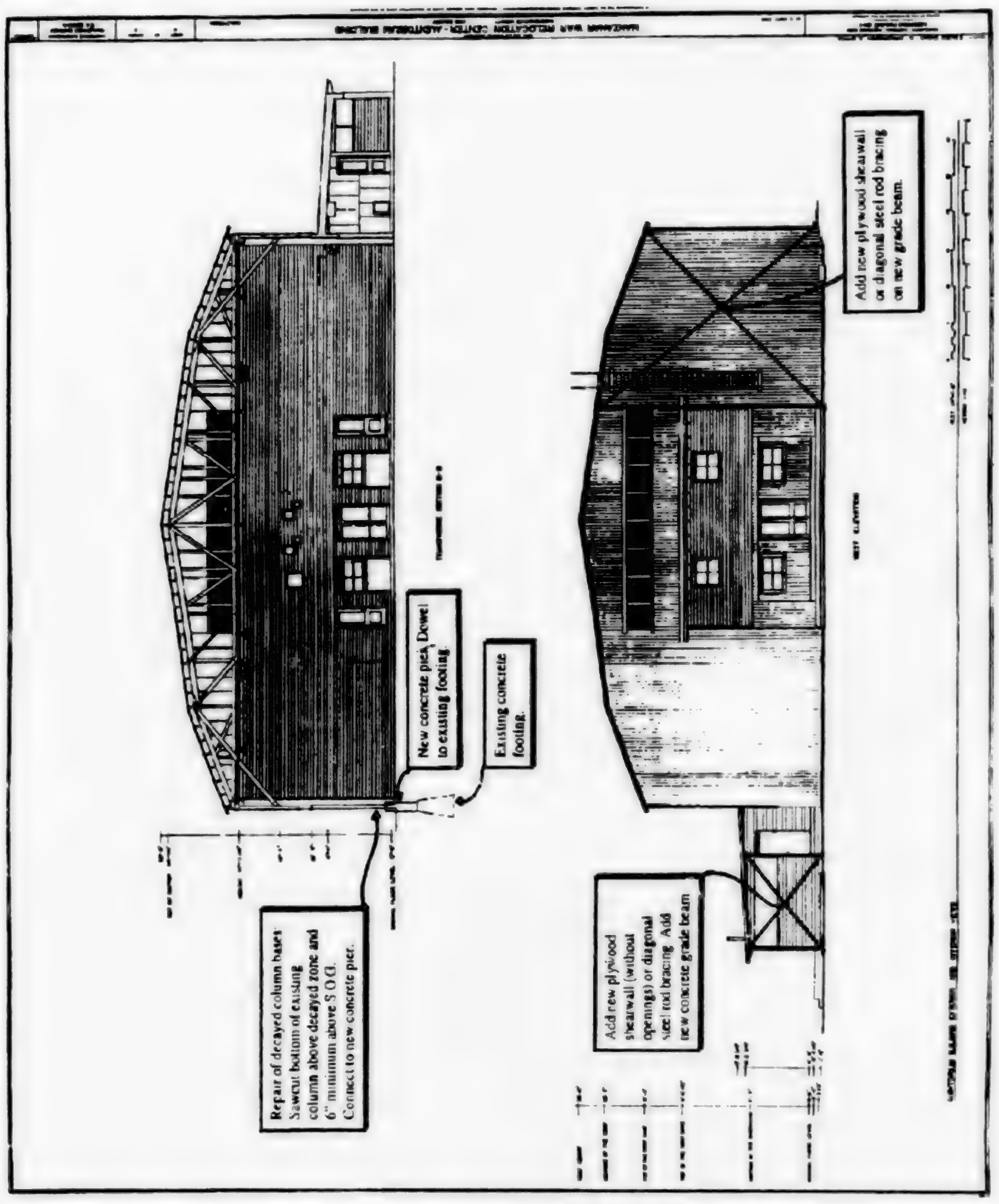
Repair of decayed column bases. Sawcut bottom of existing column above decayed zone and 6" minimum above S.O.G. Connect to new concrete pier.
New concrete pier. Dowel to existing footing.
Existing concrete footing.
Add new plywood shearwall (without openings) or diagonal steel rod bracing. Add new concrete grade beam.
Add new plywood shearwall or diagonal steel rod bracing on new grade beam.

Figure 7 West Elevation Added Bracing & Repair of Decayed Columns

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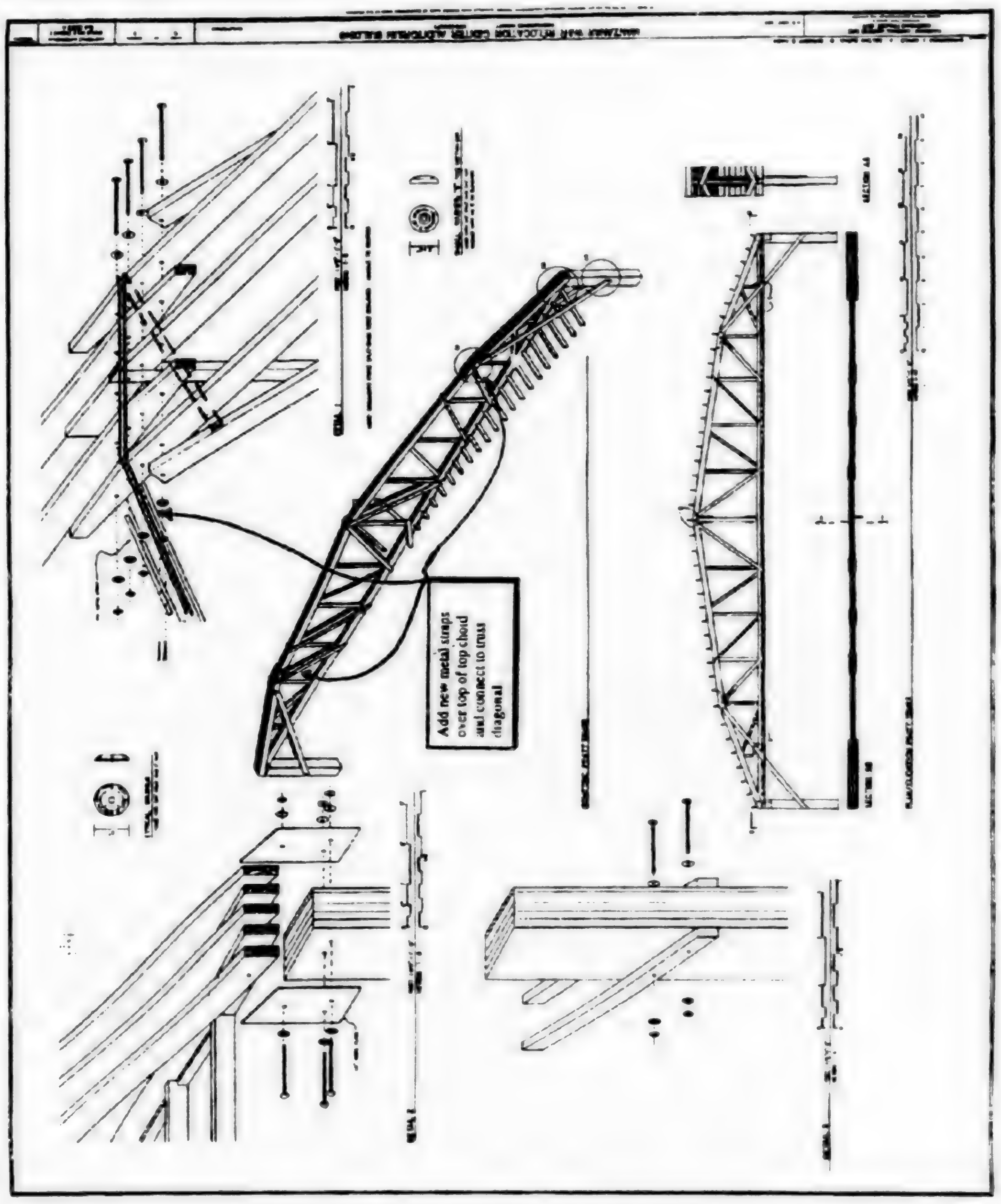
Add new metal straps over top of top chord and connect to truss diagonal

Figure 8 Repair of Cracked Roof Trusses

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Photo No. 1
East Elevation

Photo No. 2
North and East Elevations

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Photo No. 3
South and west Elevations

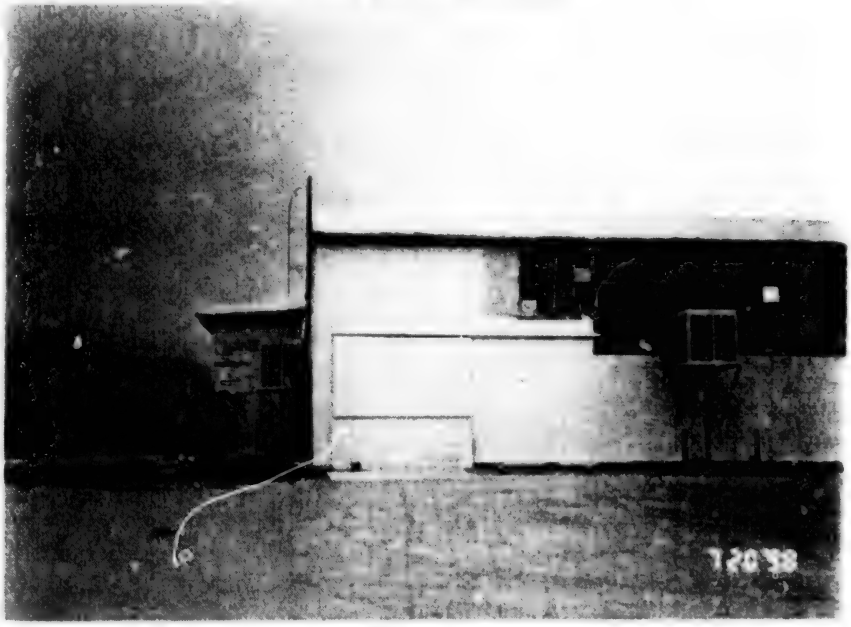

Photo No. 4
Large Added Door Opening in South Wall

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Consulting Engineers
[illegible] Kipling, P.O. Box [illegible]
[illegible] Ridge, Colorado [illegible]

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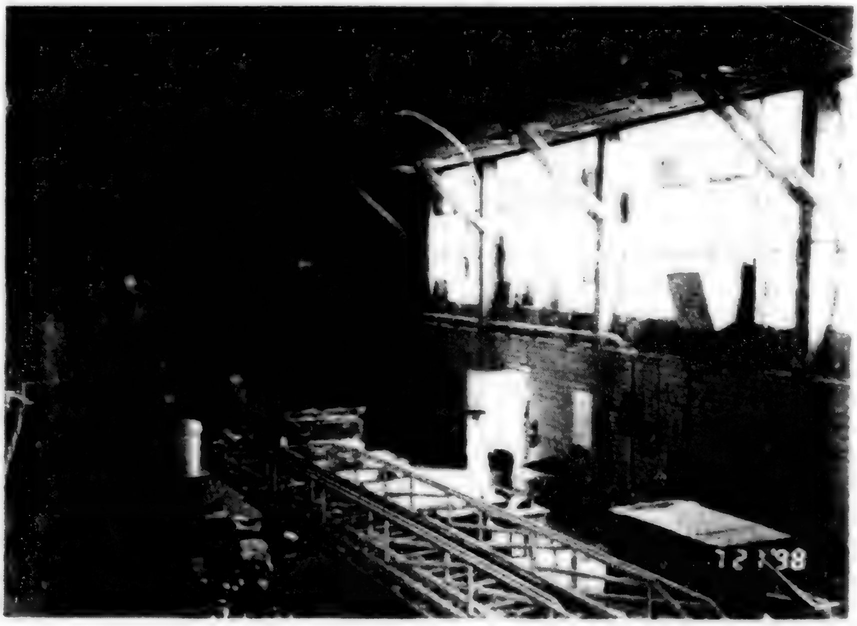

Photo No. 5
Knee Braces Attached to Wood Columns, South Wall

Photo No. 6
Wall Openings on Grid A at Grid 7

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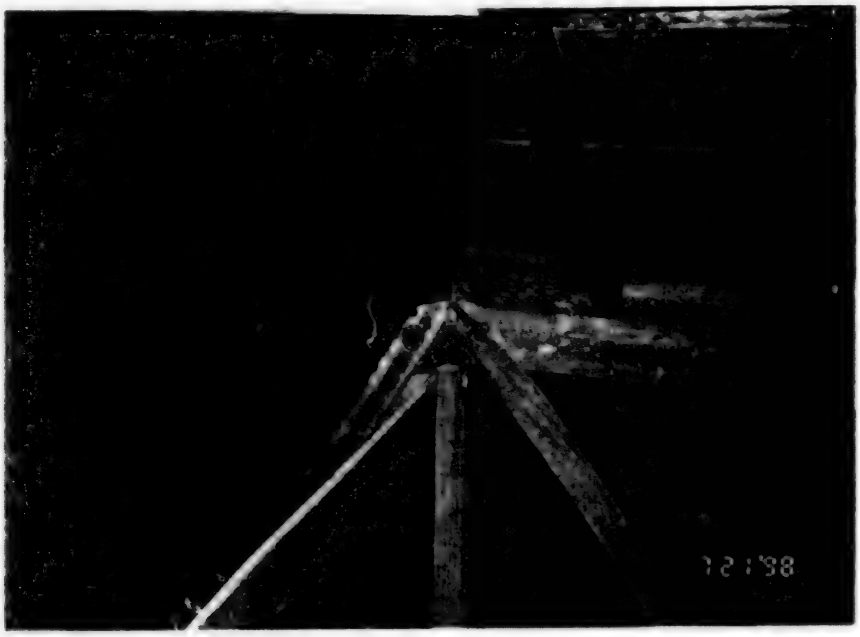

Photo No. 7
Wood Roof Framing in Auditorium

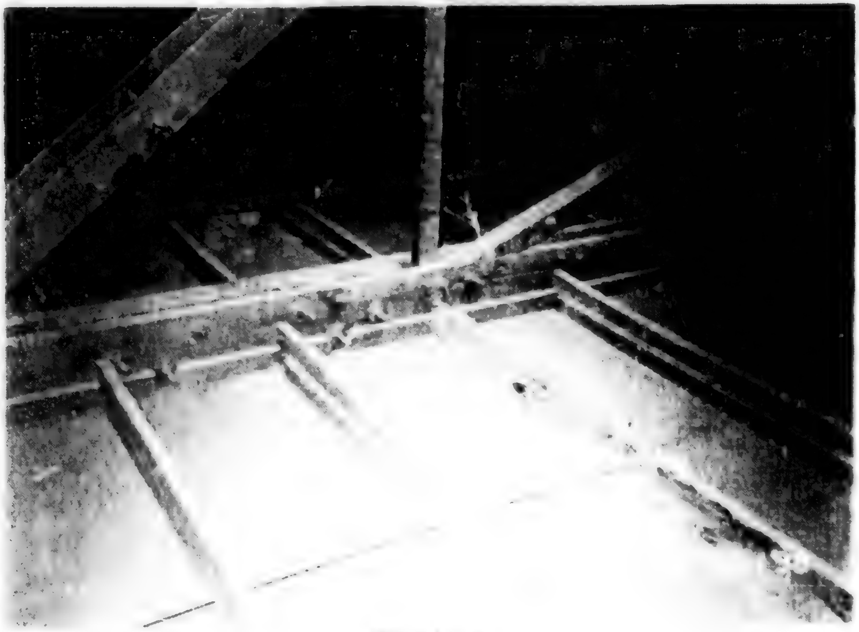

Photo No. 8
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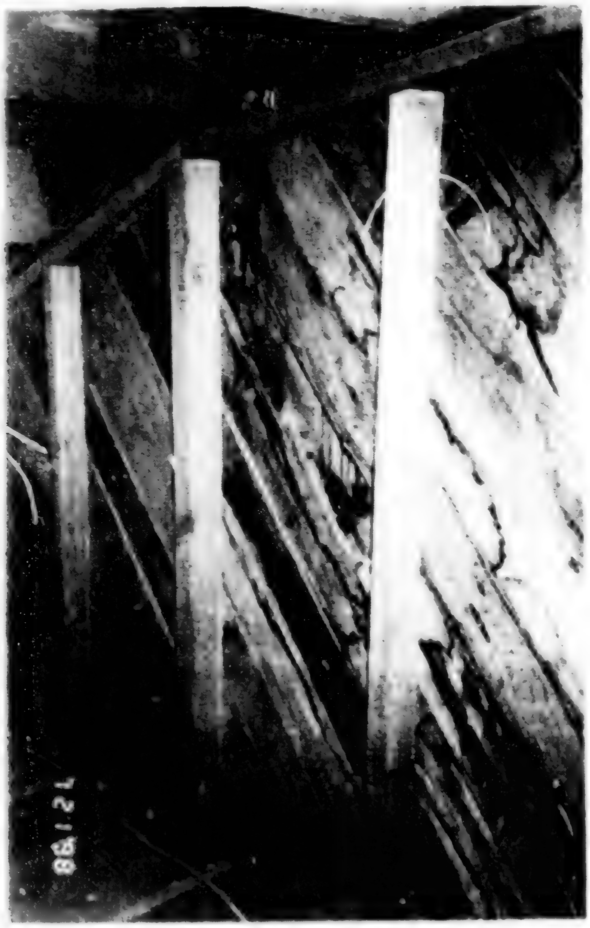

Photo No. 11
Typical Interior Wall Framing Showing Diagonal Wall Sheathing

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Photo No. [illegible]

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Photo No. [illegible]

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Photo No. 13
Bottom of South Column at Grids A-4

Photo No. 14
[illegible]

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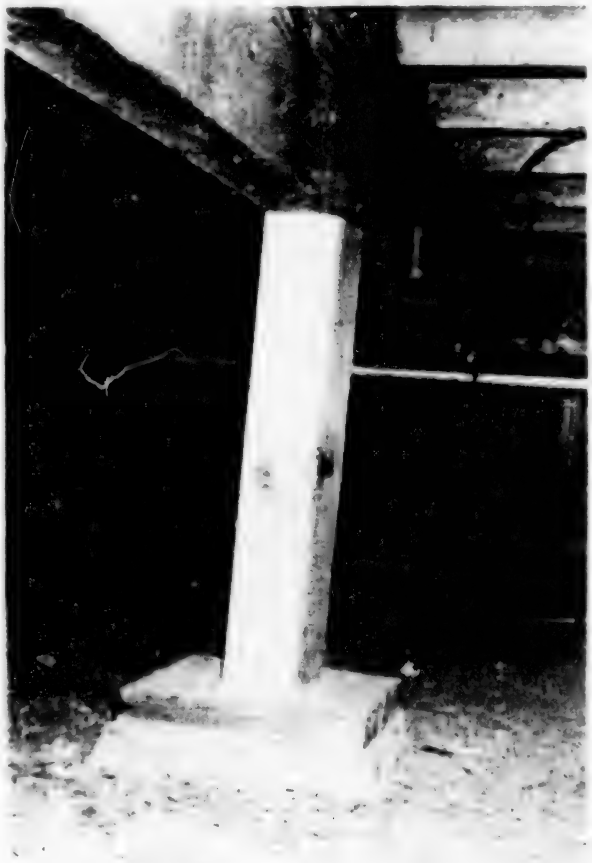

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Photo No. 17
Unbraced Cripple Wall at North Wing

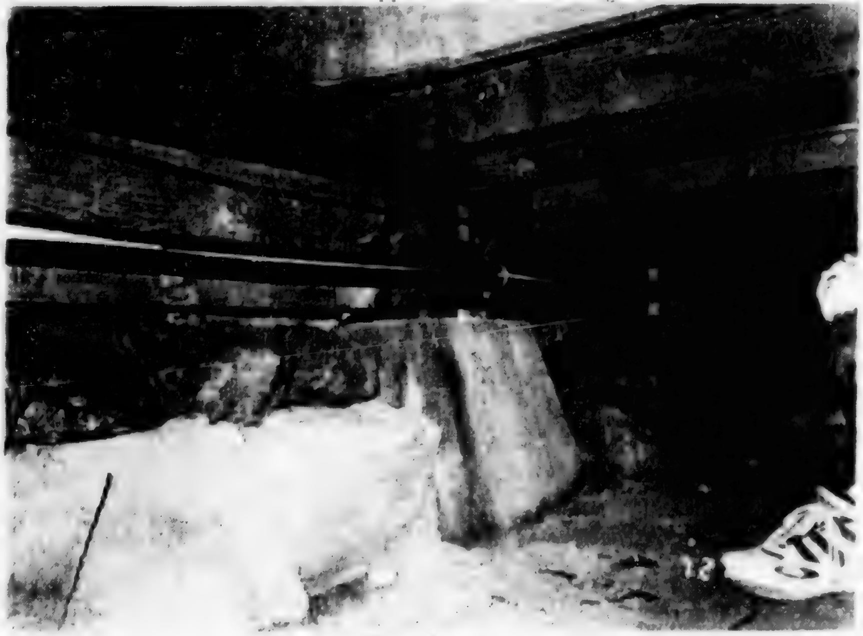

Photo No. [illegible]
Tapered Footing Supporting Wood Column at [illegible]

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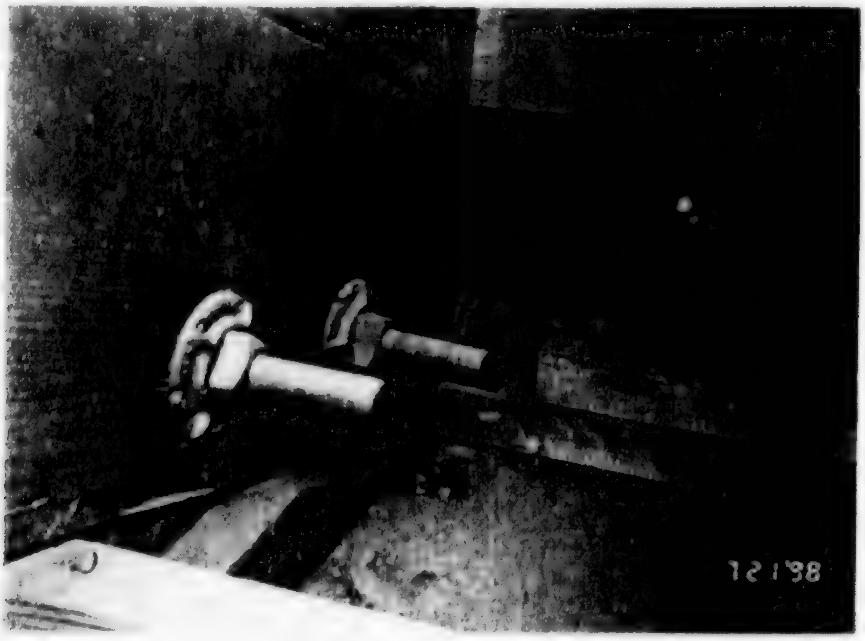

Photo No. 19
[illegible] Plate [illegible] at Truss to Column Connections

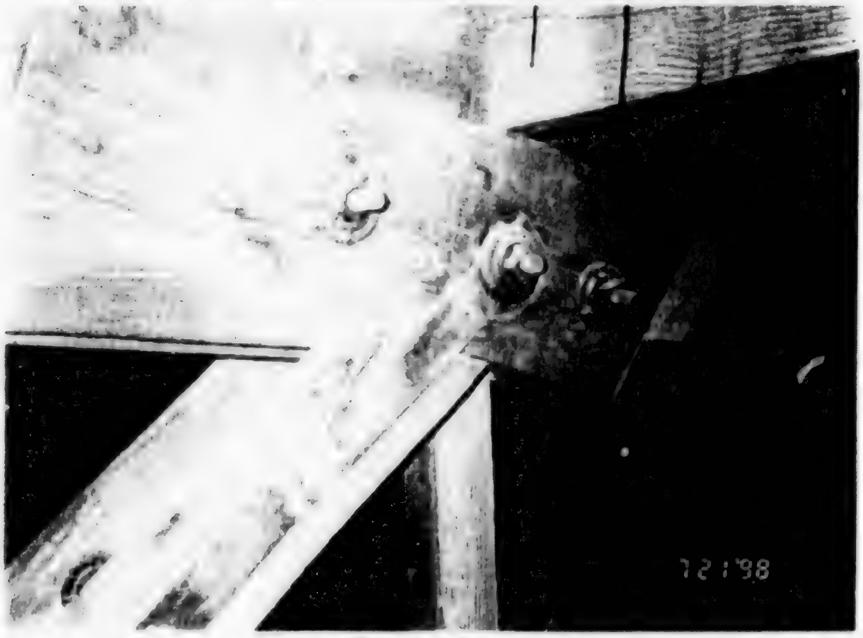

Photo No. 20
[illegible] Connection

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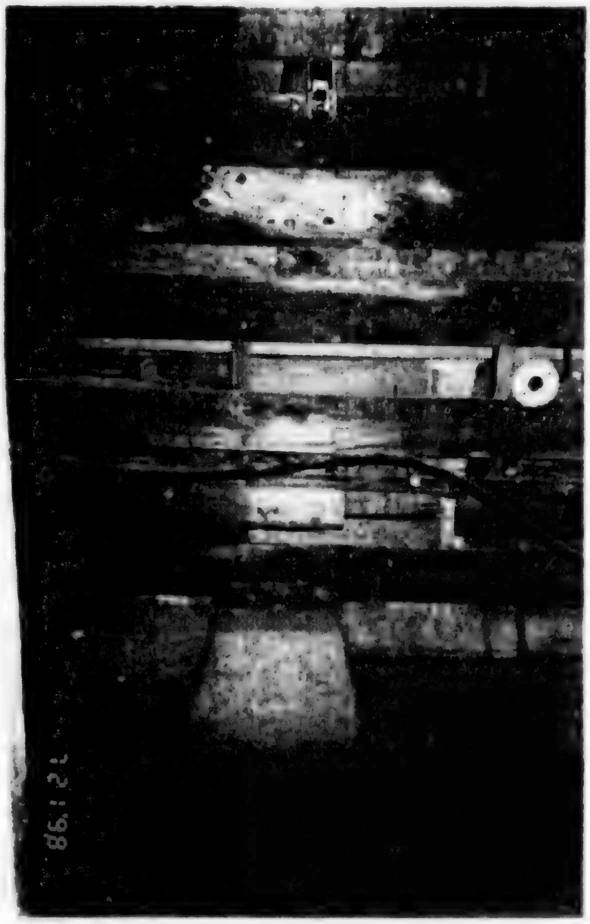

Photo No. 2[illegible]

Auditorium East Wall and Roof [illegible]

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FEMA 178 / JUNE 1992
DATA SUMMARY SHEET
MANZANAR AUDITORIUM BUILDING

BUILDING DATA

Year built: 1944 Year(s) remodeled: 1954
Date of Evaluation: July 1998
Area, (sq. ft.): 14,700 Length: 126' Width: 100'

CONSTRUCTION DATA

Roof framing: Straight 1x wood sheathing on 2x wood framing at 24" on center on wood trusses in the auditorium, and straight 1x wood sheathing on 2x wood rafters in the north, east, and west wing.

Intermediate floor framing: Wood flooring on 1x diagonal wood sheathing on 2x wood framing at 12" or 16" on center.

Ground floor Auditorium: Slab-on-grade. Basement: Not applicable.

Ground floor North Wing: Wood framing over 1x diagonal wood sheathing on 2x wood framing on wood beams and posts.

Exterior walls: Wood lap siding on diagonal 1x redwood sheathing or horizontal pine sheathing on 2x wood framing at 16 or 24" on center.

Openings: Numerous windows and large door openings.

Columns: Built-up wood. Foundations: Cast-in-place shallow spread footings.

General condition of structure: Fair.

Evidence of settling: None.

LATERAL FORCE RESISTING SYSTEM

	East-west	North-south
Model building type:	Wood bearing wall system	Wood bearing wall system
Building period T:	0.15 seconds	0.17 seconds

Unreduced base shear,

$$V = [(08\ 80A_v \times S)/(R \times T^{2/3})] \times (W) \text{ or } V = [2.12A_a/R] \times W$$
$$= 13\%\ W$$
$$= 33 \text{ kips}$$

	East-west	North-south
Response Modification Coefficient, R:	6.5	6.5

EVALUATION DATA

$A_a = 0.4$ $A_v = 0.4$

Site soil profile type: S_2 Site soil coefficient, $S = 1.2$

FEMA 178 / JUNE 1992
EVALUATION STATEMENTS FOR BUILDING TYPE 2: WOOD, COMMERCIAL AND INDUSTRIAL

BUILDING SYSTEMS

T [✓ F] N/A LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation.

[✓ T] F N/A REDUNDANCY: The structure will remain laterally stable after the failure of any single element.

T [✓ F] N/A VERTICAL DISCONTINUITIES: All shear walls, infilled walls, and frames are continuous to the foundation.

T [✓ F] N/A DETERIORATION OF WOOD: None of the wood members shows signs of decay, shrinkage, splitting, fire damage, or sagging and none of the metal accessories is deteriorated, broken, or loose.

[✓ T] F N/A OVERDRIVEN NAILS: There is no evidence of overdriven nails in the shear walls or diaphragms.

SHEAR WALLS

T [✓ F] N/A SHEARING STRESS CHECK: The building satisfies the Quick Check of the shearing stress in wood shear walls.

T [✓ F] N/A OPENINGS: Walls with garage doors or other large openings are braced with plywood shear walls or are supported by adjacent construction through substantial positive ties.

T [✓ F] N/A WALL REQUIREMENTS: All walls supporting tributary areas of 24 to 100 square feet per foot of wall are plywood sheathed with proper nailing or rod braced and have a height-to-depth (*H/D*) ratio of 1 to 1 or less or have properly detailed and constructed hold-downs.

T [✓ F] N/A CRIPPLE WALLS: All exterior cripple walls below the first floor level are braced to the foundation with shear elements

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Manzanar Auditorium Building
Appendix A
December, 1998

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DIAPHRAGMS

T F [✓ N/A] REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings that are larger that 50 percent of the building width in either plan dimension.

T [✓ F] N/A SHEATHING: None of the diaphragms consist of straight sheathing or have span/depth ratios greater and 2 to 1.

T [✓ F] N/A SPANS: All diaphragms with spans greater than 24 feet have plywood or diagonal sheathing.

T F [✓ N/A] UNBLOCKED DIAPHRAGMS: Unblocked wood panel diaphragms consist of horizontal spans less than 40 feet and have span/depth ratios less than or equal to 3 to 1.

T [✓ F] N/A SPAN/DEPTH RATIO: If the span/depth ratios of wood diaphragms are greater than 3 to 1, there are nonstructural walls connected to all diaphragm levels at less than 40-foot spacing.

T [✓ F] N/A DIAPHRAGM CONTINUITY: None of the diaphragms are composed of split-level floors or have expansion joints.

T [✓ F] N/A CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.

CONNECTIONS

T [✓ F] N/A WOOD SILLS: All wall elements are bolted to the foundation sill at 6-foot spacing or less with proper edge distances for concrete and wood.

T [✓ F] N/A WOOD POSTS: There is positive connection of the posts to the foundation and the elements being supported.

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Manzanar Auditorium Building
Appendix A
December, 1998

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FEMA 178 / JUNE 1992 EVALUATION STATEMENTS FOR FOUNDATIONS AND GEOLOGIC SITE HAZARDS

CONDITION OF FOUNDATIONS

[✓ T] F N/A	FOUNDATION PERFORMANCE: The structure does not show evidence of excessive foundation movement such as settlement or heave that would affect its integrity or strength.
[✓ T] F N/A	DETERIORATION: There is no evidence that foundation elements have deteriorated due to corrosion, sulphate attack, material breakdown, or other reasons in a manner that would affect the integrity or strength of the structure.

CAPACITY OF FOUNDATIONS

T [✓ F] N/A	OVERTURNING: The ratio of the effective horizontal dimension, at the foundation level of the seismic resisting system, to the building height (base/height) exceeds 1.4A_v.
T [✓ F] N/A	TIES BETWEEN FOUNDATION ELEMENTS: Foundation ties adequate for seismic forces exist where footings, piles, and piers are not restrained by beams, slabs, or competent soils or rock.
T F [✓ N/A]	LATERAL FORCE ON DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil.
T F [✓ N/A]	POLE BUILDINGS: Pole foundations have adequate embedment.
[✓ T] F N/A	SLOPING SITES: The grade difference from one side of the building to another does not exceed one-half story.

GEOLOGIC SITE HAZARDS

T F N/A [✓ Unknown]	LIQUEFACTION: Liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 feet under the building.
[✓ T] F N/A	SLOPE FAILURE: The building site is sufficiently remote from potential earthquake induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating small predicted movements without failure.
T F N/A [✓ Unknown]	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated.

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FEMA 178 / June 1992 Seismic Evaluation
Manzanar Auditorium Building

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Appendix B1, Load Sheets

December, 1998

Location	Description		Unit Weight (psf)
Roof Loads			
Auditorium Roof Dead Loads			
Uniform Loads:	New Rolled asphalt roofing		2
	Assumed original asphalt roofing		2
	Insulation (assume 1" rigid under roofing)		1
	1 x wood sheathing		3
	2 x roof joists at 24" o.c.		2
			10 psf
Ceiling Loads:	2 x ceiling joists at 24" o.c.		1
	Celotex ceiling panels		0.5
	Lights		0.5
	Sprinkler piping		1.5
			3.5 psf
Roof Truss Weight	Top Chord:		
	3x9x16'	96 lbs	
	2-2x9x16'	96	
	2-3x9x27'	324	
		516	
	Bottom Chord		
	2-3x9x40'	480 lbs	
	Web Verticals:		
	2-3x4x3'	12 lbs	
	1-3x4x6'	12	
	1-3x6x7'	25	
	1-3x4x7'	15	
	1-3x4x8'	17	
	1/2-3x6x9'	16	
		97	
	Web Diagonals		
	2-3x6x18'	123 lbs.	
	1-3x4x8	17	
	2-2x6x9'	36	
	2-2x4x10'	26	
	1-3x4x10'	20	
	2-2x6x11'	44	
		266	
	Truss Bridging		
	1/2-2-2x8x24'	65 lbs.	
	Truss Columns:		
	3-3x16x21'	630 lbs.	
	2-2x16x21'	210	
		840	
	Half Truss Weight =	2264 lbs	
	Truss Weight per foot =	68 plf	
	Truss weight smeared =		4 psf
	Total Auditorium roof load =		**18 psf**

*Snow loads less than 30 psf are negligible in seismic analysis per FEMA 178 Sect. 2.4.2.

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Manzanar Auditorium Building

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Appendix B1, Load Sheets

December, 1998

North, East, and West Wing Roof Dead Loads	Uniform Loads.	
	Same as Auditorium Roof	10 psf

Floor Loads

North Wing Floor over crawl space	Wood flooring (3/4" x 3 1/4" T & G)	4 psf
	1 x diagonal wood subfloor	3
	2x6 at 12" o.c.	2
	6 x 10 beams at 10' o.c.	2
		11 psf
East Wing floor at Civil Defense Room and old Stage floor	Wood flooring (3/4" x 3 1/4" T & G)	4 psf
	1 x diagonal wood subfloor	3
	2x12 at 12" o.c.	4
	1/2" gypsum board ceiling	2
	Lights and misc.	2
		15 psf
Projection room floor in West Wing	2" concrete topping	20 psf
	1 x diagonal wood subfloor	3
	2x16 at 16" o.c.	1
	1/2" gypsum board ceiling	2
	Lights and misc.	1
		27 psf

Wall Loads

Typical exterior walls	3/4" exterior lap siding	2 psf
	1 x nominal diagonal redwood sheathing	3
	3/4" interior lap siding	2
	2 x framing at 16 " o. c.	1
		8 psf
Typical interior wall	3/4" exterior lap siding	2 psf
	3/4" interior lap siding	2
	2 x framing at 16 " o. c.	1
		5 psf

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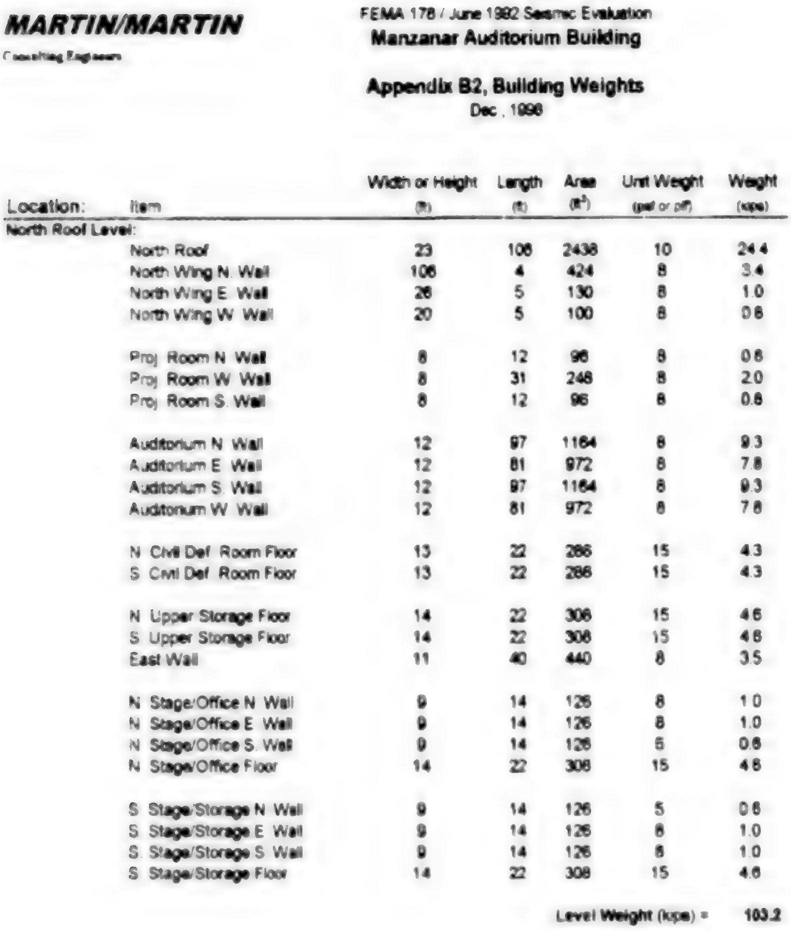

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Appendix B2, Building Weights

Dec., 1998

Location:	Item	Width or Height (ft)	Length (ft)	Area (ft^2)	Unit Weight (psf or plf)	Weight (kips)
North Roof Level:						
	North Roof	23	106	2438	10	24.4
	North Wing N. Wall	106	4	424	8	3.4
	North Wing E. Wall	26	5	130	8	1.0
	North Wing W. Wall	20	5	100	8	0.8
	Proj. Room N. Wall	8	12	96	8	0.8
	Proj. Room W. Wall	8	31	248	8	2.0
	Proj. Room S. Wall	8	12	96	8	0.8
	Auditorium N. Wall	12	97	1164	8	9.3
	Auditorium E. Wall	12	81	972	8	7.8
	Auditorium S. Wall	12	97	1164	8	9.3
	Auditorium W. Wall	12	81	972	8	7.8
	N. Civil Def. Room Floor	13	22	286	15	4.3
	S. Civil Def. Room Floor	13	22	286	15	4.3
	N. Upper Storage Floor	14	22	308	15	4.6
	S. Upper Storage Floor	14	22	308	15	4.6
	East Wall	11	40	440	8	3.5
	N. Stage/Office N. Wall	9	14	126	8	1.0
	N. Stage/Office E. Wall	9	14	126	8	1.0
	N. Stage/Office S. Wall	9	14	126	5	0.6
	N. Stage/Office Floor	14	22	308	15	4.6
	S. Stage/Storage N. Wall	9	14	126	5	0.6
	S. Stage/Storage E. Wall	9	14	126	8	1.0
	S. Stage/Storage S. Wall	9	14	126	8	1.0
	S. Stage/Storage Floor	14	22	308	15	4.6

Level Weight (kips) = 103.2

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Appendix B3, Base Shear

Coeficients:

$A_v =$ 0.4 [NEHRP Map #3. 1994 Edition]
$A_a =$ 0.4 [NEHRP Map #4. 1994 Edition]
$S =$ 1.2 [FEMA 178, Table 2.1 - Soil Profile Type S2]
$R =$ 6.5 [FEMA 178, Table 2.4.3.1 - *Bearing Wall-Light Framed Walls w/Shear Panels*]

Periods:

$T_a = (0.05 * h_n)/\mathrm{sqrt}(L)$ [FEMA 178. Sec. 2.4.3.2, Method 1c.]

North-South Direction
$L =$ 100.0 ft [Building Dimension in N-S Direction]
$h_n =$ 33.0 ft [Building Height]

$T_a =$ 0.165 seconds

East-West Direction
$L =$ 127.0 ft [Building Dimension in N-S Direction]
$h_n =$ 33.0 ft [Building Height]

$T_a =$ 0.146 seconds

Seismic Design Coef.:

$C_s = (0.80 * A_v * S)/(R * T^{2/3})$ [FEMA 178. Sec. 2.4.3.1, Eq. 2-4]

$C_{s\ max} = (2.12 * A_a)/R$ [FEMA 178. Sec. 2.4.3.1, Eq. 2-5]

North-South Direction
$C_s =$ 0.20
$C_{s\ max} =$ 0.13

$C_s =$ 0.13 [Use lesser of above]

East-West Direction
$C_s =$ 0.21
$C_{s\ max} =$ 0.13

$C_s =$ 0.13 [Use lesser of above]

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Appendix B3, Base Shear

Base Shear:

$V = C_s * W$ [FEMA 178, Sec. 2.4.3.1, Eq. 2-3]

North-South Direction
$V =$ 32.6 kips

East-West Direction
$V =$ 32.6 kips

Vertical Distribution Factors:

$C_{vx} = w_x * h_x^k / \mathrm{sum}(w_i * h_i^k)$ [FEMA 178, Sec. 2.4.3.7, Eq. 2-9]

$k =$ 1.0 [for buildings with $T < 0.5$ sec]

$w_2 =$ 147.1 kips [weight of upper roof level]
$w_1 =$ 103.2 kips [weight of north roof level]

$h_2 =$ 29.0 feet [mean height of upper roof level]
$h_1 =$ 12.0 feet [mean height of north roof level]

$C_{v2} =$ 0.775
$C_{v1} =$ 0.225

Base Shear Vertical Distribution:

$F_x = C_{vx} * V$ [FEMA 178, Sec. 2.4.3.7, Eq. 2-8]

North-South Direction
$F_2 =$ 25.3 kips
$F_1 =$ 7.3 kips

East-West Direction
$F_2 =$ 25.3 kips
$F_1 =$ 7.3 kips

GLOSSARY OF TERMS

BRACED FRAME is essentially a vertical truss designed to resist lateral forces parallel to the plane of the braced frame. Lateral forces are resisted by axial tension and compression on the frame members.

CHORD is the boundary element of a diaphragm or shear wall that resists axial forces due to bending of the diaphragm.

DIAPHRAGM is a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical elements.

DRAG STRUT (Collector) is a member or element of a diaphragm provided to transfer lateral forces from the diaphragm to the vertical elements of the lateral system (shear walls or bracing).

MOMENT FRAME in an assemblage of beams and columns connected to resist lateral forces in the plane of the frame by bending the beams and columns.

SHEAR WALL is a wall designed to resist lateral forces parallel to the plane of the wall.

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NPS D-6, April 1999

United States Department of the Interior • National Park Service

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